THAMES RIVER BASIN

WATERFORD, CONNECTICUT

MILLER POND DAM
00154

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST 1980

UNCLASSIFIED

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19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Thames River Basin
Waterford, Conn.

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The dam has total length of approximately 425 ft. and consists of an embankment section with upstream and downstream masonry faces and a masonry spillway section. The dam is 19.5 ft. in height. Based upon the visual inspection at the site and past performance, the project is judged to be in poor condition. Miller Pond Dam is classified as a high hazard, small size dam. The test flood range to be considered is from one-half to full PMF.

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BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam:	MILLER POND DAM
Inventory Number:	CT 00154
State Located:	CONNECTICUT
County Located:	NEW LONDON
Town Located:	WATERFORD
Stream:	HUNTS BROOK
Owner:	HERBERT SCHACHT
Date of Inspection:	MARCH 20, 1980
Inspection Team:	PETER HEYNEN, P.E.
	MURALI ATLURU, P.E.
	MIRON PETROVSKY
	THEODORE STEVENS

The dam, built in the 1870's, has a total length of approximately 425 feet and consists of an embankment section with upstream and downstream masonry faces and a masonry spillway section ~~(See Sheet B-1).~~ The top of the embankment section, at elevation 83.5±, varies in width from approximately 14 to 40 feet and is 3.5 feet above the spillway crest. The dam is 19.5 feet in height above the old streambed of Hunts Brook and, with the pond level to the top of the dam, impounds approximately 700 acre-feet of water. The spillway is an 87.8 foot long broad-crested weir located at the right end of the dam and is founded on bedrock. A 4' x 4.5' masonry high-level outlet culvert through the spillway section, at invert elevation 74.0±, is located near the right abutment of the spillway. A 2'x3' masonry low-level outlet culvert, at invert elevation 64.0±, is located in the earthfill section of the dam.

Based upon the visual inspection at the site and past performance, the project is judged to be in poor condition. There are areas which require monitoring and/or maintenance such as: seepage at several locations on the downstream face and toe of the dam, seepage of the low-level outlet culvert, the inoperable low-level outlet gate, eroded areas on the top of the dam, deteriorated masonry at several locations on the dam, and possible erosion or undermining due to high velocity flows along the downstream toe of the spillway and dam.

In accordance with the Army Corps of Engineers' Guidelines, Miller Pond Dam is classified as a high hazard, small size dam. The test flood range to be considered is from one-half to full Probable Maximum Flood (PMF). The test flood for Miller Pond Dam is equivalent to the ½ PMF. Peak inflow to the reservoir at the ½ PMF is 8,610 cubic feet per second (cfs); peak outflow is 7,730 cfs with the dam overtopped by 2.7 feet. The spillway capacity with the reservoir level to the top of the dam is 1,610 cfs, which is equivalent to 21% of the routed test flood outflow.

It is recommended that the owner retain the services of a registered professional engineer to perform a more detailed hydraulic analysis of the adequacy of the existing project discharge Other items of importance are grading of the top of the dam to eliminate eroded areas, repair of the low-level outlet gate, repair of deteriorated masonry, inspection of the toe of the spillway and dam during no flow conditions and determination of the significance of all seepage. Recommendations made by the engineer should be implemented by the owner.

The above recommendations and further remedial measures presented in Section 7 should be instituted within one year of the owner's receipt of this report.

Peter M. Heynen
Project Manager - Geotechnical
Cahn Engineers, Inc.

STATE OF CONNECTICUT
PETER M. HEYNEN
NO. 9618
REGISTERED
PROFESSIONAL ENGINEER

C. Michael Horton
Department Head
Cahn Engineers, Inc.

STATE OF CONNECTICUT
CLARENCE M. HORTON
No. 8433
REGISTERED
PROFESSIONAL ENGINEER

This Phase I Inspection Report on Miller Pond Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and are hereby submitted for approval.

ARAMAST MAHTESIAN, Member
Geotechnical Engineering Branch
Engineering Division

CARNEY M. TERZIAN, Member
Design Branch
Engineering Division

RICHARD DIBUONO, Chairman
Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions there of. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

The Phase I Investigation does not include an assessment of the need for fences, gates, no-trespassing signs, repairs to existing fences and railings and other items which may be needed to minimize trespass and provide greater security for the facility and safety to the public. An evaluation of the project for compliance with OSHA rules and regulations is also excluded.

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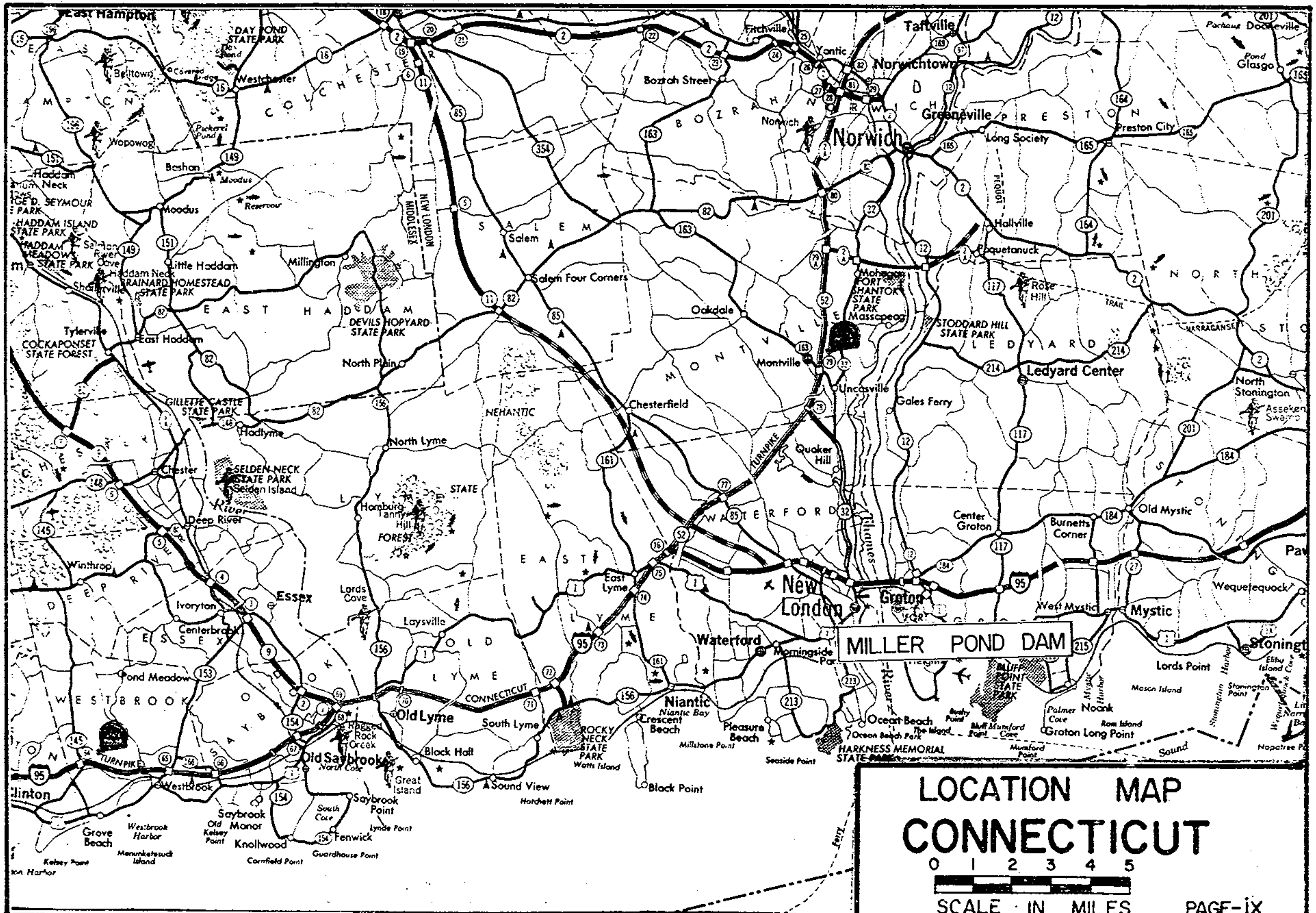
(February, 1980)

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS. CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER	NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS	Miller Pond Dam Hunts Brook	Waterford CONNECTICUT	DATE May 1980 CE # 27 785 KA PAGE viii


MILLER POND DAM
LOCATION MAP
CONNECTICUT
0 1 2 3 4 5
SCALE IN MILES
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East Hampton
Norwich
Norwichtown
Taftville
Greeneville
Preston City
Long Society
Colchester
Westchester
Salem
Salem Four Corners
Oakdale
Montville
Uncasville
Chesterfield
Quaker Hill
Waterford
New London
Groton
Mystic
West Mystic
Old Mystic
Ledyard Center
Gales Ferry
Center Groton
Burnetts Corner
North Stonington
Niantic
East Lyme
Old Lyme
South Lyme
Black Hall
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Moodus
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Haddam Neck
Millington
North Plain
Tylerville
Winthrop
Bozrah Street
Yantic
Fitchville
Ocean Beach
Pleasure Beach
Crescent Beach
Black Point
Noank
Groton Long Point
Lords Point
Stonington
Wequetequock
DEVILS HOPYARD STATE PARK
GILLETTE CASTLE STATE PARK
SELDEN NECK STATE PARK
ROCKY NECK STATE PARK
HARKNESS MEMORIAL STATE PARK
BLUFF POINT STATE PARK
FORT SHANTOK STATE PARK
STODDARD HILL STATE PARK
DAY POND STATE PARK
COCKAPONSET STATE FOREST
Thames River
Connecticut
Long Island Sound

PHASE I INSPECTION REPORT

MILLER POND DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of April 14, 1980 from William E. Hodgson, Jr. Colonel, Corps of Engineers. Contract No. DACW 33-80-C-0052 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dam.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on Hunt's Brook in a rural area of the Town of Waterford, County of New London, State of Connecticut. The dam is shown on the U.S.G.S. Montville Quadrangle Map having coordinates latitude N41°24.4' and longitude W72°07.9'.

b. Description of Dam and Appurtenances - As shown on Sheet B-1, the 20 foot high dam is a masonry and earthfill gravity structure, probably founded on bedrock for its entire length. The project is approximately 425 feet in length, consisting of an approximately 335 foot long dogleg shaped earthfill section with upstream and downstream vertical masonry faces and an 87.8 foot long masonry spillway. There is a high-level outlet through the spillway section and a low-level outlet through the earthfill and masonry section.

The 87.8 foot long spillway, at the right end of the dam, is a broad-crested masonry weir of trapezoidal cross-section with a shallow, gravelly approach channel and a nearly vertical downstream face. The spillway discharges onto a large expanse of exposed bedrock at the toe of the dam.

The earthfill and masonry section has a maximum height of approximately 19.5 feet and a top elevation 3.5 feet above the spillway crest. It is approximately 14 feet wide near its left end, widening to a maximum of approximately 40 feet near its center.

A metal sluice gate controls flow through a 2'x3' masonry low-level culvert through the embankment; however, there is no mechanism with which to raise the gate. An approximately 4'x4.5' high-level outlet with upstream masonry training walls is located near the right end of the spillway. There is no gate or operating mechanism for this outlet; however, there are slots in the training walls in which stoplogs may be placed.

c. Size Classification - SMALL - The dam impounds 700 acre-feet of water with the lake level to the top of the dam, which is 19.5 feet above the old streambed. According to the U.S. Army Corps of Engineers' Recommended Guidelines, a dam with this storage capacity is classified as small in size.

d. Hazard Classification - HIGH - If the dam were breached, there is potential for loss of more than a few lives and extensive property damage to at least 3 houses approximately 8 feet above the streambed of Hunt's Brook in a rural area off of Bloomingdale Road approximately 3700 feet downstream of the dam. A secondary impact area, where 3 more structures including 2 houses, would be affected by a breach of the dam, is approximately 6,200 feet from the dam (See Sheet D-1).

e. Ownership - Mr. Herbert Schacht
Hunts Brook Rd.
Waterford, Ct. 06385
Tel.: (203) 443-8074 (Home)
(203) 442-9454 (Office)

The dam was originally built and owned by the Miller family. The Schacht family acquired the property in 1931.

f. Operator - Mr. Herbert Schacht (See above)

g. Purpose - The wooded area around the pond is used for recreational purposes by the Waterford Country Day School.

h. Design and Construction History - The following information is believed to be accurate, based on the available data and correspondence and an interview with the owner of the dam. The dam was constructed around 1873 to supply water to a downstream factory. There is no record of any alterations or repairs to the dam until 1963, at which time the low-level outlet gate was repaired, trees and brush on the dam and at its base were removed, the masonry faces of the dam were repointed and dead trees were removed from the spillway.

i. Normal Operational Procedures - The low-level outlet for the dam is kept in a closed position and the high-level outlet is kept open. No formal operational procedures exist.

1.3 PERTINENT DATA

a. Drainage Area - The drainage area is 10.5 square miles of relatively undeveloped, rolling terrain.

b. Discharge at Damsite - Discharge is over the spillway, through the high-level outlet in the spillway section and through the low-level outlet in the masonry and earthfill section.

1. Outlet Works (Conduits):	
4'x4.5' masonry culvert at invert el. 74.0±	240± cfs (pond level at top of dam)
2'x3' masonry culvert at invert el. 64.0±	200± cfs (pond level at top of dam)
2. Maximum flood at damsite:	Not known
3. Ungated spillway capacity at top of dam el. 83.5±:	1,610 cfs
4. Ungated spillway capacity at test flood el. 86.2:	3,800 cfs

5. Gated spillway capacity at normal pool: N/A

6. Gated spillway capacity at test flood: N/A

7. Total spillway capacity at test flood el. 86.2: 3,800 cfs

8. Total project discharge at top of dam el. 83.5: 2,050 cfs

9. Total project discharge at test flood el. 86.2: 7,730 cfs

c. Elevations (National Geodetic Vertical Datum based on assumed spillway crest elevation of 80.0)

1. Streambed at toe of dam:	64.0±
2. Maximum tailwater:	N/A
3. Upstream portal invert diversion tunnel:	N/A
4. Recreation pool:	N/A
5. Full flood control pool:	N/A
6. Spillway crest (ungated):	80.0 (assumed datum)
7. Design Surcharge (Original):	Not known
8. Top of dam:	83.5±
9. Test flood surcharge:	86.2

d. Reservoir Length

1. Normal pool:	3,400± ft.
2. Flood control pool:	N/A
3. Spillway crest pool:	3,400± ft.
4. Top of dam pool:	3,500± ft.
5. Test flood pool:	3,600± ft.

e. Reservoir Storage

1. Normal pool:	410± acre-ft.
2. Flood control pool:	N/A

3. Spillway crest pool: 410± acre-ft.

4. Top of dam pool: 700± acre-ft.

5. Test flood pool: 950± acre-ft.

f. <u>Reservoir Surface</u>

1. Normal pool: 77± acres

2. Flood control pool: N/A

3. Spillway crest pool: 77± acres

4. Top of dam pool: 90± acres

5. Test flood pool: 99± acres

g. <u>Dam</u>

1. Type: Masonry and earthfill

2. Length: 425± ft.

3. Height: 19.5± ft.

4. Top width: Varies 40± ft. max.
14± ft. min.

5. Side slopes: Vertical (Upstream)
Vertical (Downstream)

6. Zoning: N/A

7. Impervious Core: Not known

8. Cutoff: Not known

9. Grout Curtain: N/A

10. Other: N/A

h. <u>Diversion and Regulatory Tunnel</u> - N/A

i. <u>Spillway</u>

1. Type: Broad-crested masonry weir

2. Length of weir: 87.8 ft.

3. Crest elevation: 80.0 (assumed datum)

4. Gates:	N/A
5. Upstream Channel:	Shallow, gravel bottom
6. Downstream Channel:	Exposed bedrock
7. General:	Downstream face is at slight batter

j. Regulating Outlets - The outlets are a high-level outlet through the spillway section and a low-level outlet through the masonry and earthfill section.

High-Level Outlet

1. Invert:	74.0$\pm$
2. Size:	4'x4.5'
3. Description:	Masonry culvert
4. Control mechanism:	None
5. Other:	Slots for stop logs

Low-Level Outlet

1. Invert:	64.0$\pm$
2. Size:	2'x3'
3. Description:	Masonry culvert
4. Control Mechanism:	None in place
5. Other:	Gate in closed position

SECTION 2: ENGINEERING DATA

2.1 DESIGN DATA

The available data consists of inventory data by the State of Connecticut, correspondence concerning the 1963 repairs to the dam, and drawings of the 1963 repairs by W.A. Morse, Civil Engineer (See Appendix B).

The drawings and correspondence indicate the design features stated previously in this report.

2.2 CONSTRUCTION DATA

The available data consists of an inspection report by B. H. Palmer for the Connecticut Water Resources Commision concerning the 1963 repairs (Page B-6).

2.3 OPERATIONS DATA

Lake level readings are not taken. It is not known if the spillway capacity of the dam has ever been exceeded. No formal operations records are known to exist.

2.4 EVALUATION OF DATA

a. Availability - Available data was provided by the State of Connecticut; Chandler, Palmer and King, Engineers and the owner. The owner made the project available for visual inspection.

b. Adequacy - The limited amount of detailed engineering data available was generally inadequate to perform an in-depth assessment of the dam, therefore, the final assessment of this dam must be based primarily on visual inspection, performance history, hydraulic computations of spillway capacity and hydrologic estimates.

c. Validity - A comparison of record data and visual observations reveals no significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 FINDINGS

a. General - The general condition of the project is poor. The inspection revealed several areas requiring maintenance, repair and monitoring. At the time of the inspection, the pond level was at elevation 77.8, i.e. 5.7 feet below the top of the dam with water flowing through the high-level spillway outlet.

b. Dam

Top of Dam - The top of the embankment is grass covered with a heavy growth of brush and large trees. Towards the right end of the top of the dam, adjacent to the spillway, an eroded area approximately 6 feet by 6 feet and 6 inches in depth was noted (Photo 1). From this area, erosion of the earthfill extends to the spillway along the upstream masonry wall, which is severely damaged and displaced.

Upstream Face - The upstream masonry face of the dam is in fair condition. The northern (left) area of this face is covered by brush and trees growing along the shoreline upstream of the dam. There are open and cracked mortar joints on the masonry face.

Downstream Face - There is extensive seepage and a large wet area near the toe of the downstream face at a distance of approximately 50 feet to the left of the low-level outlet. The seepage was flowing both through the masonry joints and probably from the base of the dam. The general seepage flow rate in this zone was about 30 gallons per minute (gpm), or more, with separate leaks of up to 10 gpm (Photo 3). In this area, many mortar joints were cracked and leached. The toe of the dam is covered by heavy brush and large trees. One wet area was encountered at a higher elevation than the area described above and had a flow rate at about 1 gpm.

Spillway - The masonry spillway crest is in good condition. No visible cracks or deteriorated zones were observed on the crest (Photo 2). Substantial tree growth and wood debris were noted on the upstream slope of the spillway (Photo 4). The downstream face had some cracking in the mortar joints and severalseeps, with a total flow of approximately 3 gpm, in the area of the high-level outlet.

No visible deterioration of the almost submerged high-level spillway outlet was noted. The upstream stone training walls of the outlet are damaged, with partially displaced and fallen stones (Photo 5).

The spillway discharge channel is of exposed bedrock and does not have distinct limits. Approximately one-half of the spillway discharge was running along the toe of the spillway and dam with high velocities, and could cause erosion or undermining along the toe.

c. Appurtenant Structures - The sluice gate stem of the low-level outlet culvert through the earthfill masonry dam is broken and the sluice gate, presently in a closed position, is not operable. However, a considerable flow (approximately 30 gpm) through the culvert, was observed at its outlet (Photo 6). Most of the flow observed at the outlet is entering the culvert from the surrounding body of the dam.

d. Reservoir - The area surrounding the pond is generally wooded and undeveloped except for the Connecticut Turnpike which is adjacent to the northwestern shore of the pond.

e. Downstream Channel - The downstream channel is the natural streambed of Hunts Brook.

3.2 EVALUATION

Based upon the visual inspection, the project is assessed as being in poor condition. The following features which could influence the future condition and/or stability of the project were identified.

1. Significant seepage through the foundation and the masonry, accompanied by leaching of the cement mortar joints, could weaken the masonry and create stability problems.

2. Constant high velocity flow through the high-level outlet may be causing erosion of its upstream training walls.

3. The high velocity flow running along the downstream toe of the spillway and the dam could lead to deterioration and undermining of the masonry at the toe.

4. The lack of an operable mechanism for the sluice gate does not permit use of the low-level outlet in emergency situations.

5. The trees growing on the crest and masonry faces of the dam and on the upstream slope of the spillway can cause weakness of the masonry and additional seepage by penetration of tree roots.

6. Seepage from the body of the dam into the low-level outlet culvert could threaten the stability of the dam due to a loss of soil from the body of the embankment.

SECTION 4: OPERATIONAL AND MAINTENANCE PROCEDURES

4.1 OPERATIONAL PROCEDURES

a. General - Lake level readings are not taken and no regulating procedures are followed at the dam.

b. Description of Any Warning System in Effect - No formal warning system is in effect. The owner reports that he is at the dam during large storms and calls local officials if he detects a problem.

4.2 MAINTENANCE PROCEDURES

a. General - There is no formal program of maintenance or inspection of the dam; however, the owner does perform periodic informal inspections.

b. Operating Facilities - No formal program for maintenance of operating facilities is in effect. The low-level outlet gate was last operated in 1963.

4.3 EVALUATION

The operation and maintenance procedures are generally poor. A formal program of operations and maintenance procedures should be implemented, including documentation to provide complete records for future reference. Also, a formal warning system should be developed and implemented within the time frame indicated in Section 7.1c. Remedial operation and maintenance recommendations are presented in Section 7.3.

SECTION 5: EVALUATION HYDRAULIC/HYDROLOGIC FEATURES

5.1 GENERAL

The watershed is 10.5 square miles of mostly wooded rolling terrain and is sparsely developed. The dam is located on Hunts Brook and has an 87.8 ft. long stone masonry spillway to the right. The spillway section has a high-level outlet with invert elevation 74.0 and the dam section has a low-level outlet with invert elevation 64.0. The high-level outlet has no gate and the low-level outlet is inoperable. The storage of the project is estimated to be 410 acre-feet with the pond level at the spillway crest and 700 acre-feet with the pond level at the top of the dam.

5.2 DESIGN DATA

No hydraulic or hydrologic design data or computations could be found for the original construction.

5.3 EXPERIENCE DATA

The maximum discharge at the dam site is not known and no information was found to indicate that there have been any problems (including overtopping) arising at the dam.

5.4 VISUAL OBSERVATIONS

The spillway is founded on rock and the discharge section immediately downstream of the structure has some obstructions such as boulders, brush and a tree; however, these conditions would have very little effect on the hydraulic performance of the dam.

5.5 TEST FLOOD ANALYSIS

Based upon the U.S. Army Corps of Engineers "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978, the watershed classification (Rolling) and the watershed area of 10.5 square miles, a Probable Maximum Flood (PMF) of 17,200 cubic feet per second (cfs) or 1640 cfs per square mile is estimated at the damsite. In accordance with the size (small) and hazard (high) classification, the range of test floods to be considered is from the ½ PMF to the PMF. Based upon the severity of the downstream hazard, the test flood for Miller Pond Dam is equivalent to the ½ PMF. Assuming the pond level at the spillway crest at the beginning of the test flood, peak inflow is 8,610 cfs; peak outflow is 7,730 cfs with the dam overtopped by 2.7 feet. The spillway capacity to the top of the dam is 1610 cfs which is equivalent to 21% of the routed test flood outflow (Appendix D-10, D-11).

5.6 DAM FAILURE ANALYSIS

The dam failure analysis is based on the April, 1978 Army Corps of Engineers "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs." With the reservoir level at the top of the dam, peak prefailure outflow would be about 1860 cfs and the peak failure outflow from a breach of the dam would total about 12,000 cfs. Based on an examination of the conditions downstream of the dam, it is assumed that attenuation of the flood volume would be insignificant and hence the peak flow rate at the impact areas is taken as 12,000 cfs in this analysis.

A breach of the dam would result in a rise of 5.2 feet in the water level of the stream at the initial impact area, located 3700 feet downstream of the dam in the vicinity of Bloomingdale Road. This corresponds to an increase in the water level of the stream from a prefailure flow depth of 5.1 feet to a depth of 10.3 feet after failure of the dam. This condition, in conjunction with the culvert constriction, would impact 3 houses. One house, located upstream of the culvert and north of the stream, is approximately 8 feet above the channel bed and its first floor would be flooded with approximately 2.3 feet of water. Two additional houses east of Bloomingdale Road would also be impacted by 2 feet of floodwater. A secondary impact area 6200 feet downstream of the dam in the vicinity of Old Norwich Road would similarly be impacted by breaching of the dam, with flooding of at least 3 buildings, one of which contains several businesses. The rise in the stage of the stream just above the Old Norwich Road is estimated to be 4.8 feet, which corresponds to an increase from a prefailure flow depth of 4.8 feet to a depth of 9.6 after failure of the dam. The building containing businesses is approximately 7 feet above the channel bed and would be flooded with 2.6 feet of water. Also, two houses located east of Old Norwich Road and adjacent to the Brook are likely to be impacted by dam failure. Because a breach of Miller Pond Dam would cause severe economic loss and the loss of more than a few lives, it is classified as a high hazard dam.

SECTION 6: EVALUATION OF STRUCTURAL STABILITY

6.1 VISUAL OBSERVATIONS

The visual inspection did not reveal any indications of immediate stability problems. There are areas of seepage, deterioration, and erosion, as described in Section 3, however they are not considered stability concerns at the present time.

6.2 DESIGN AND CONSTRUCTION DATA

The drawings and data available and listed in Appendix B were not sufficient to perform an in-depth stability analysis of the dam. No engineering assumptions, data or calculations could be found for the original design of the dam.

6.3 POST CONSTRUCTION CHANGES

The post-construction changes of the project include the following data pertaining to the 1963 repairs to the dam.

1. Operating mechanism of the sluice gate of the low-level outlet.

2. Repointing the cement mortar joints of the masonry faces of the dam.

6.4 SEISMIC STABILITY

The project is in Seismic Zone 1 and according to the recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 PROJECT ASSESSMENT

a. Condition - Based upon the visual inspection of the site and past performance, the project appears to be in fair condition. No evidence of immediate structural instability was observed in the embankments, spillway and appurtenant structures. However, there are areas which require maintenance, repair and monitoring.

Based upon the Army Corps of Engineers' "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978, the watershed classification and hydraulic/hydrologic computations, peak inflow to the pond at test flood is 12,000 cubic feet per second (cfs); peak outflow is 12,000 cfs with the dam overtopped 4.7 feet. Based upon our hydraulic computations, the spillway capacity to the top of dam is 1,900 cfs, which is equivalent to approximately 16% of the routed test flood outflow.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the project must be based solely on visual inspection, past performance and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within one year of the owner's receipt of this report.

7.2 RECOMMENDATIONS

It is recommended that further studies be made by a registered professional engineer qualified in dam design and inspection pertaining to the following items. Recommendations made by the engineer should be implemented by the owner.

1. A detailed hydraulic analysis of the adequacy of the project discharge and existing outlet facilities, including an evaluation of the outlet culvert through the right section of the dam and the absence of a low-level outlet.

2. An inspection of the inside of the masonry arch culvert and the sluice gate openings through the right embankment of the dam for possible deterioration and an inspection of the outlet canal, its masonry training wall and 12 inch C.I. drain pipe to determine their condition. These inspections can be performed during the annual draining of the canal.

3. An inspection of the masonry spillway and spillway apron when no water is flowing over the spillway. This should include evaluation of seepage through the spillway, possible deterioration of the masonry downstream face of the spillway and possible undermining or erosion conditions at the toe.

6. Determination of the origin and significance of seepage at the downstream face and the toe of the dam and, if necessary, development of a boring program to determine the condition of the masonry of the dam and spillway and foundation conditions.

7. Based upon the findings of item 6, above, a program to monitor or eliminate seepage through the dam, spillway and foundation should be developed.

8. Repair of the leached and open mortar joints on the masonry of the upstream and downstream faces of the dam and spillway.

7.3 REMEDIAL MEASURES

Operation and Maintenance Procedures - The following measures should be undertaken by the owner within the length of time indicated in Section 7.1.c, and continued on a regular basis.

1. Round-the-clock surveillance should be provided during periods of heavy precipitation or high project discharge. A formal downstream warning system should be developed to be used in case of emergencies at the dam.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference.

3. A comprehensive program of inspection by a registered professional engineer qualified in dam inspection should be instituted on an annual basis.

4. The top of the masonry walls of the dam with displaced and fallen masonry blocks should be reinforced and restored.

5. Eroded areas of the earthfill dam crest should be filled with suitable soils, compacted and seeded.

6. The damaged masonry of the upstream training walls of the high-level outlet should be repaired.

7. The cutting of grass, brush and trees on the crest, faces and at the toe of the dam and spillway should be performed and continued as part of the routine maintenance procedures.

7.4 ALTERNATIVES

This study had identified no practical alternatives to the above recommendations.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST

PARTY ORGANIZATION

PROJECT Miller Pond Dam

DATE: Mar 20, 1980

TIME: 2:00 pm

WEATHER: Sunny, 50°

W.S. ELEV. 77.8± U.S. ______ DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. Peter Heynen	PH	Geotechnical
2. Miron Petrovsky	MP	Geotechnical
3. Theodore Stevens	TS	Geotechnical
4. Murali Atluru	MA	Hydraulics
5. Moshé Norman	MN	Survey
6. Timothy Kavanaugh	TK	Survey

PROJECT FEATURE	INSPECTED BY	REMARKS
1. Earthfill Embankment	PH, MP, TS, MA	Fair Condition
2. Low-level Outlet Culvert	PH, MP, TS, MA	Very Poor Condition
3. High-Level Outlet	PH, MP, TS, MA	Fair Condition
4. Masonry Spillway	PH, MP, TS, MA	Fair Condition
5.		
6.		
7.		
8.		
9.		
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

PROJECT Miller Pond Dam DATE 3-20-80

PROJECT FEATURE Earthfill Embankment BY PH, MP, TS, MA

AREA EVALUATED		CONDITION
DAM EMBANKMENT		
Crest Elevation		83.5±
Current Pool Elevation		77.8±
Maximum Impoundment to Date		Not known
Surface Cracks		None observed
Pavement Condition		Grass covered
Movement or Settlement of Crest		Depression on U/s edge near low-level outlet
Lateral Movement		Too irregular to judge
Vertical Alignment		
Horizontal Alignment		
Condition at Abutment and at Concrete Structures		Fair
Indications of Movement of Structural Items on Slopes		N/A
Trespassing on Slopes		No slopes-trespassing on top
Sloughing or Erosion of Slopes or Abutments		None observed
Rock Slope Protection-Riprap Failures		N/A
Unusual Movement or Cracking at or Near Toes		None observed, but are high velocity flows along toe
Unusual Embankment or Downstream Seepage		Yes- from area near low-level outlet
Piping or Boils		None observed
Foundation Drainage Features		N/A
Toe Drains		N/A
Instrumentation System		N/A

PERIODIC INSPECTION CHECK LIST

Page A-3

PROJECT Miller Pond Dam DATE 3-20-80

PROJECT FEATURE Low-Level Outlet BY PH, MP, TS, MA

AREA EVALUATED		CONDITION
OUTLET WORKS-TRANSITION AND CONDUIT		2'X3' Masonry Culvert
General Condition of ~~Concrete~~ Masonry		Poor - Heavy Leakage
Rust or Staining on ~~Concrete~~ Masonry		None observed
Spalling		N/A
Erosion or Cavitation		N/A
Cracking		N/A
Alignment of Monoliths		N/A
Alignment of Joints		N/A
Numbering of Monoliths		N/A
		Seepage (±30 gpm) is from body of dam into culvert
		Low-level intake submerged-could not observe

PERIODIC INSPECTION CHECK LIST

PROJECT Miller Pond Dam DATE 3-20-80

PROJECT FEATURE High-Level Outlet BY PH, MP, TS, MA

AREA EVALUATED		CONDITION
OUTLET WORKS-INTAKE CHANNEL AND INTAKE STRUCTURE		High-level outlet at right end of dam in spillway section Flowing at time of inspection
a) Approach Channel		Bedrock Approach Channel
Slope Conditions		Shallow slope
Bottom Conditions		Bedrock
Rock Slides or Falls		No
Log Boom		N/A
Debris		No
Condition of ~~Concrete~~ Masonry Lining		Fair, Deterioration of right U/S training wall
Drains or Weep Holes		N/A
b) Intake Structure		
Condition of Concrete - Slots		Poor-cracked, missing
Stop Logs and Slots		Not in place-have not been for several years

PROJECT Miller Pond Dam DATE 3-20-80

PROJECT FEATURE Masonry Spillway BY PH, MP, TS, MA

AREA EVALUATED		CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS		
a) Approach Channel		
General Condition		Poor
Loose Rock Overhanging Channel		No
Trees Overhanging Channel		yes-growing at U/S side of spillway
Floor of Approach Channel		Gravel, Sand
b) Weir and Training Walls		Left training wall only-right side is rock abutment
General Condition of ~~Concrete~~ Masonry		Weir - Good cond. Wall - Fair cond.
Rust or Staining		None observed
Spalling		N/A
Any Visible Reinforcing		N/A
Any Seepage or Efflorescence		Slight seepage at right end
Drain Holes		No
c) Discharge Channel		
General Condition		Non-defined, bedrock
Loose Rock Overhanging Channel		No
Trees Overhanging Channel		yes-not a problem
Floor of Channel		Bedrock
Other Obstructions		No

APPENDIX B

ENGINEERING DATA AND CORRESPONDENCE

SUMMARY OF DATA AND COREESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
Sept. 6, 1963	Cuheca Realty Corp. c/o Mr. Herbert Schacht	William S. Wise Director Connecticut Water Resources Commission	Order to repair dam	B-2
Nov. 22, 1963	Herbert Schacht	William A. Morse, Civil Engineer	Sketch plans for repair of dam	B-4
Dec. 10, 1963	William P. Sander Engineer - Geologist Water Resources Commission	Benjamin H. Palmer Chandler & Palmer Engineers	Report on inspection of repairs to dam	B-6
Dec. 20, 1963	William S. Wise	Herbert Schacht	Progress of repairs dam	B-8
Nov. 19, 1964	Cuheca Realty Corp.	William S. Wise	Certificate of Approval	B-9
Oct. 24, 1964	File	Water Resources Commisssion	Inventory Data	B-10

STATE OF CONNECTICUT
WATER RESOURCES COMMISSION
STATE OFFICE BUILDING - HARTFORD 15, CONNECTICUT

September 6, 1963

Cuheca Realty Corporation
c/o Mr. Herbert Schacht
Waterford Country School
Fire Street, Quaker Hill
Waterford, Connecticut

Gentlemen:

According to the records in this office the so-called Miller's Pond Dam in the Town of Waterford is under your ownership.

Section 25-110 of the 1958 Revision of the General Statutes places under the jurisdiction of this Commission all dams, "which, by breaking away or otherwise, might endanger life or property." The Commission finds that failure of this dam would endanger life or property.

In accordance with Section 25-111 of the General Statutes this dam has been inspected and found to be in an unsafe condition. The statute states in part: . . . "If, after any inspection described herein, the Commission finds any such structure to be in an unsafe condition, it shall order the person, firm or corporation owning or having control thereof to place it in a safe condition or to remove it, and shall fix the time within which such order shall be carried out."

F I N D I N G

Based on the engineer's report covering the inspection of this dam, the Water Resources Commission finds the structure to be in an unsafe condition. It also finds that certain repairs or alterations are necessary to place the structure in a safe condition.

The repairs or alterations to be made should include but are not necessarily limited to the following items:

1. Remove all trees and brush on the dam and at the base of the dam
2. Rebuild entirely the wooden sluice gate
3. Repair the downstream face of the dam
4. Remove dead trees from the present spillway
5. Repair all leaks at the base of the dam

ORDER

In accordance with Section 25-111 of the General Statutes you are hereby ordered to make the repairs or alterations necessary to place the structure in a safe category or to remove the structure.

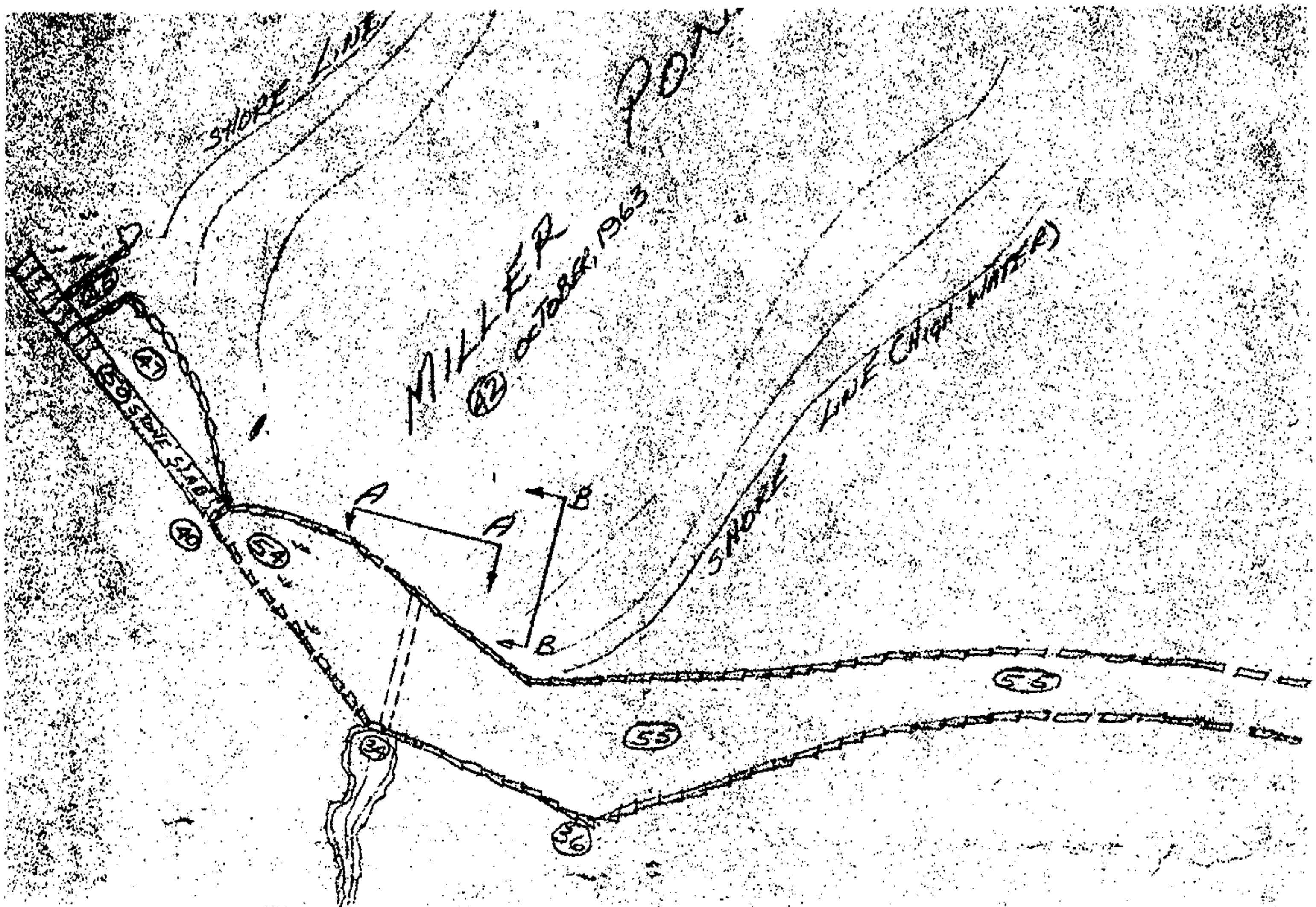
Any repairs or alterations to the structure or its removal shall be carried out in accordance with engineering plans and specifications prepared by a registered engineer and submitted to this Commission for approval and for the issuance of a permit prior to any construction on demolition work in accordance with Section 25-112 of the General Statutes.

The Commission shall be notified within two weeks what steps you plan to take to repair or remove the structure. The work shall be completed within six months of the date of this order.

Very truly yours,

William S. Wise
Director

WSW:dlp



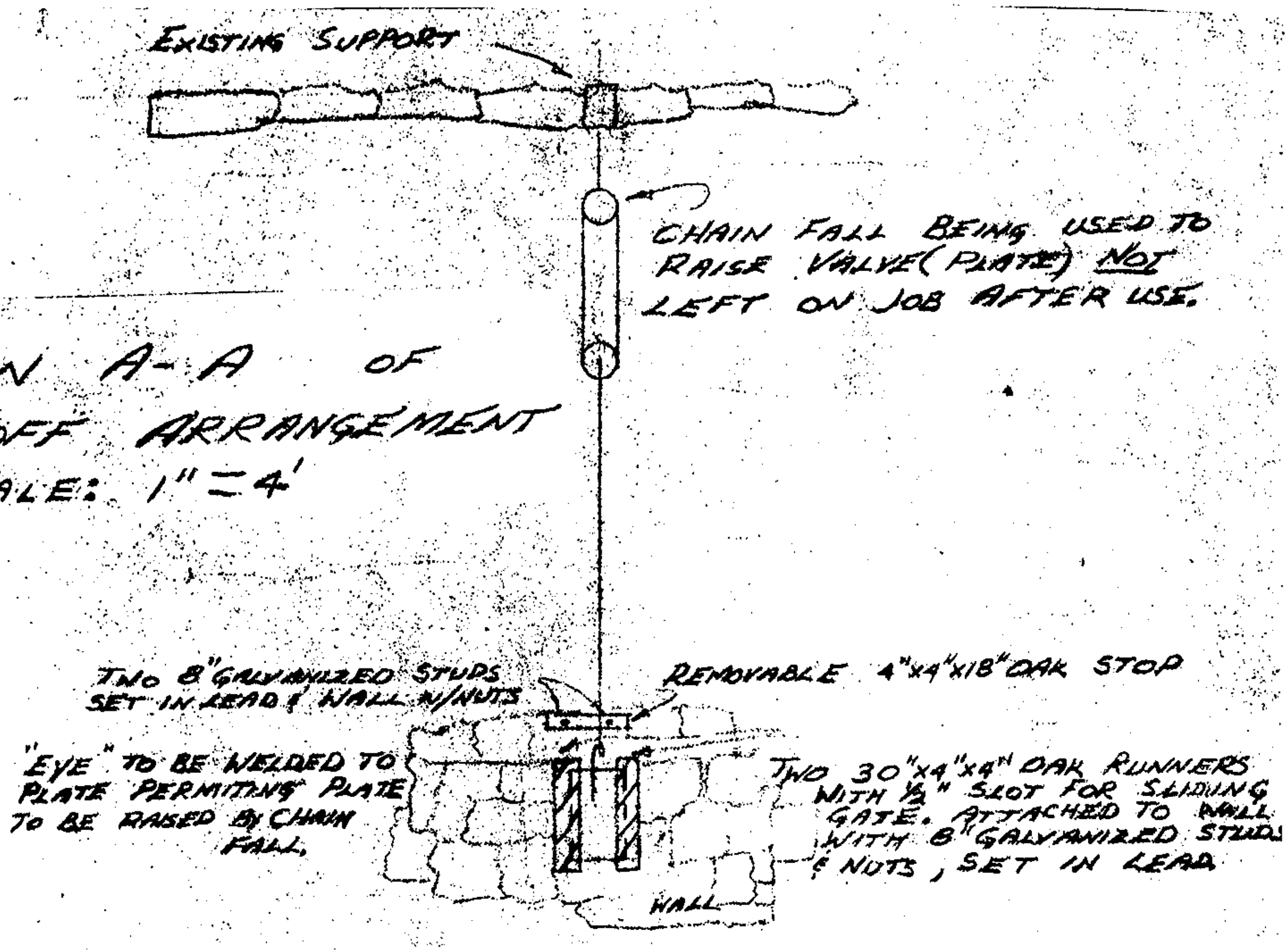
GENERAL PLAN VIEW OF DAM
SCALE: 1" = 40'

1. DAM FACES TO BE POINTED.
2. DEBRIS TO BE REMOVED FROM SPILLWAY.
3. CLAY OF ADEQUATE AMOUNT (DETERMINED BY ENGINEER) TO BE PLACED AT VALVE AREA UPSTREAM.

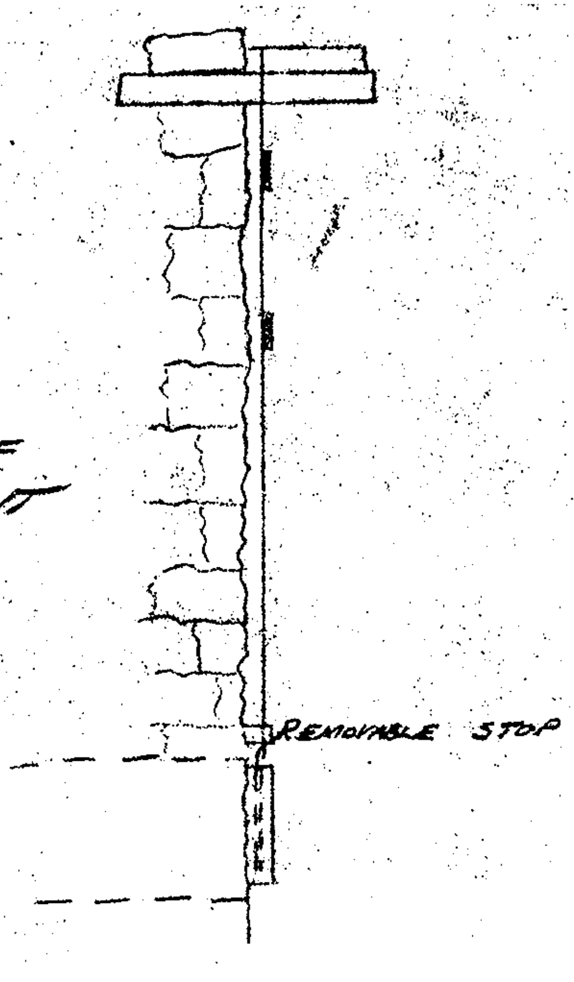
PLAN AND SECTION VIEWS
OF MILLER POND DAM
LOCATED OFF
COLCHESTER ROAD
QUAKER HILL
(WATERFORD) CONNECTICUT
~ HERBERT SCHACHT OWNER
DATE: NOV 22, 1963
W.A. MORSE ~ CIVIL ENGINEER

STATE OF CONNECTICUT
W. A. MORSE
NO. [illegible]
REGISTERED
PROFESSIONAL ENGINEER

B-4



SECTION A-A OF SHUT OFF ARRANGEMENT
SCALE: 1" = 4'



SECTION B-B OF SHUTOFF ARRANGEMENT

December 10, 1963

State of Connecticut
Water Resources Commission
State Office Building
Hartford 15, Connecticut

Re: Miller's Pond

Attention: Mr. William P. Sander
Engineer-Geologist

Dear Sir:

This is in reply to your letter dated December 4 relative to the above project. It is my opinion that a preliminary construction permit be issued on the basis of the plan which was submitted and drawn by W. A. Morse, Civil Engineer dated November 22, 1963.

Please refer to the letter to the Cuheca Realty Corporation dated September 6, 1963 and signed by W. S. Wise, Director. I visited the dam again today and give the following report.

1) All trees and brush on the dam and at the base of the dam have been cut and removed.

2) The sluice gate, as previously reported, was a metal gate and not a wooden gate. The gate was in satisfactory condition but the gate stem was disconnected. This part of the work has not been done in accordance with the plan referred to above.

3) The downstream face of the dam has been repointed in a satisfactory manner.

4) The dead trees have been removed from the present spillway.

5) The pond is pretty far down because the gate is open so that it is not possible to determine whether all of the leaks at the base of the dam have been stopped. It is my opinion that they are in better condition than at the beginning of the work.

The Contractor, Mr. Brown, told me some weeks ago that he would complete the work on the gate, but as of today it has not been completed.

Very truly yours,

CHANDLER & PALMER

B. H. Palmer

BHP/nir

Waterford Country School

R. F. D. No. 1

Quaker Hill, Connecticut

Dec. 20, 1963

William S. Wise, Director
Water Resources Commission
State of Connecticut
State Office Building
Hartford 15, Connecticut

Re: Miller's Pond Dam

Dear Mr. Wise:

Thank you for granting us permission to carry out the repairs to the Miller's Pond Dam in accordance with the plans prepared by Mr. W.A. Morse which we submitted to your office.

We have completed the following work: the trees and brush have been removed from the dam and from the base of the dam; the down stream and upstream faces of the dam have been repaired by painting the stone work with cement; the dead trees have been removed from the spillway; fill has been placed against the base of the dam.

We have not completed the rebuilding of the sluice gate and additional clay is to be placed against the base of the dam.

I am writing to you to report the progress made to date and to request a six month extension of the permit to enable us to complete the work in favorable weather. Mr. William Sander of your office advised me this morning over the telephone that a request for an extension must be made to you in writing.

Mr. Benjamin Palmer has advised us before and during all repair work. We are pleased to cooperate with Mr. Palmer in carrying out the recommendations of your commission.

The Browne Construction Co. of Quaker Hill is doing the work.

Mr. Wayne Morse of Quaker Hill is our consulting engineer.

Very truly yours,

Herbert Schacht

HS/s
cc: Mr. B. Palmer
Mr. M. Browne
Mr. W. Morse
Bd. of Selectmen, Town of Waterford

STATE OF CONNECTICUT

WATER RESOURCES COMMISSION

STATE OFFICE BUILDING · HARTFORD 15, CONNECTICUT

CERTIFICATE OF APPROVAL

November 19, 1964

Cuheca Realty Corporation
c/o Waterford Country School
Quaker Hill, Connecticut

TOWN: Waterford
RIVER: Hunts Brook
TRIBUTARY:
CODE NO.: T 6.7 HT 2.0

Gentlemen:

NAME AND LOCATION OF STRUCTURE: Millers Pond Dam, located east of Old Colchester Road in the Town of Waterford.

DESCRIPTION OF STRUCTURE AND WORK PERFORMED: Repairs to the dam in accordance with plans prepared by W. A. Morse, dated November 22, 1963.

CONSTRUCTION PERMIT ISSUED UNDER DATE OF: December 13, 1963.

This certifies that the work and construction included in the plans submitted, for the structure described above, has been completed to the satisfaction of this Commission and that this structure is hereby approved in accordance with Section 25-114 of the 1958 Revision of the General Statutes.

The owner is required by law to record this Certificate in the land records of the town or towns in which the structure is located.

WATER RESOURCES COMMISSION

BY: William S. Wise
William S. Wise, Director

No. WT-9

WATER RESOURCES COMMISSION
SUPERVISION OF DAMS
INVENTORY DATA

Inventoried By WPS

Date 28 OCTOBER 1964

Name of Dam or Pond MILLERS POND

Code No. T 6.7 HT 2.0

Nearest Street Location COLCHESTER ROAD

Town WATERFORD

U.S.G.S. Quad. MONTVILLE

Name of Stream HUNTS BROOK

Owner CUHECA REALTY CORPORATION

Address c/o WATERFORD COUNTRY SCHOOL

QUAKER HILL

ok 1/78

Pond Used For RECREATION

Dimensions of Pond: Width 1200 FEET Length 3300 FEET Area ~~100~~ ACRES

Total Length of Dam 150 FEET Length of Spillway 90 FEET

Location of Spillway WEST END OF DAM

Height of Pond Above Stream Bed 20 FEET

Height of Embankment Above Spillway 4 FEET

Type of Spillway Construction MASONRY

Type of Dike Construction MASONRY

Downstream Conditions WOODS, HOUSES

Summary of File Data ORDER DATED 9-6-63, CONSTRUCTION PERMIT DATED 12-13-63

Remarks REPAIRS TO DAM SHOULD BE FINISHED "WITHIN A FEW WEEKS".

B-10

YES

Class B

APPENDIX C

DETAIL PHOTOGRAPHS

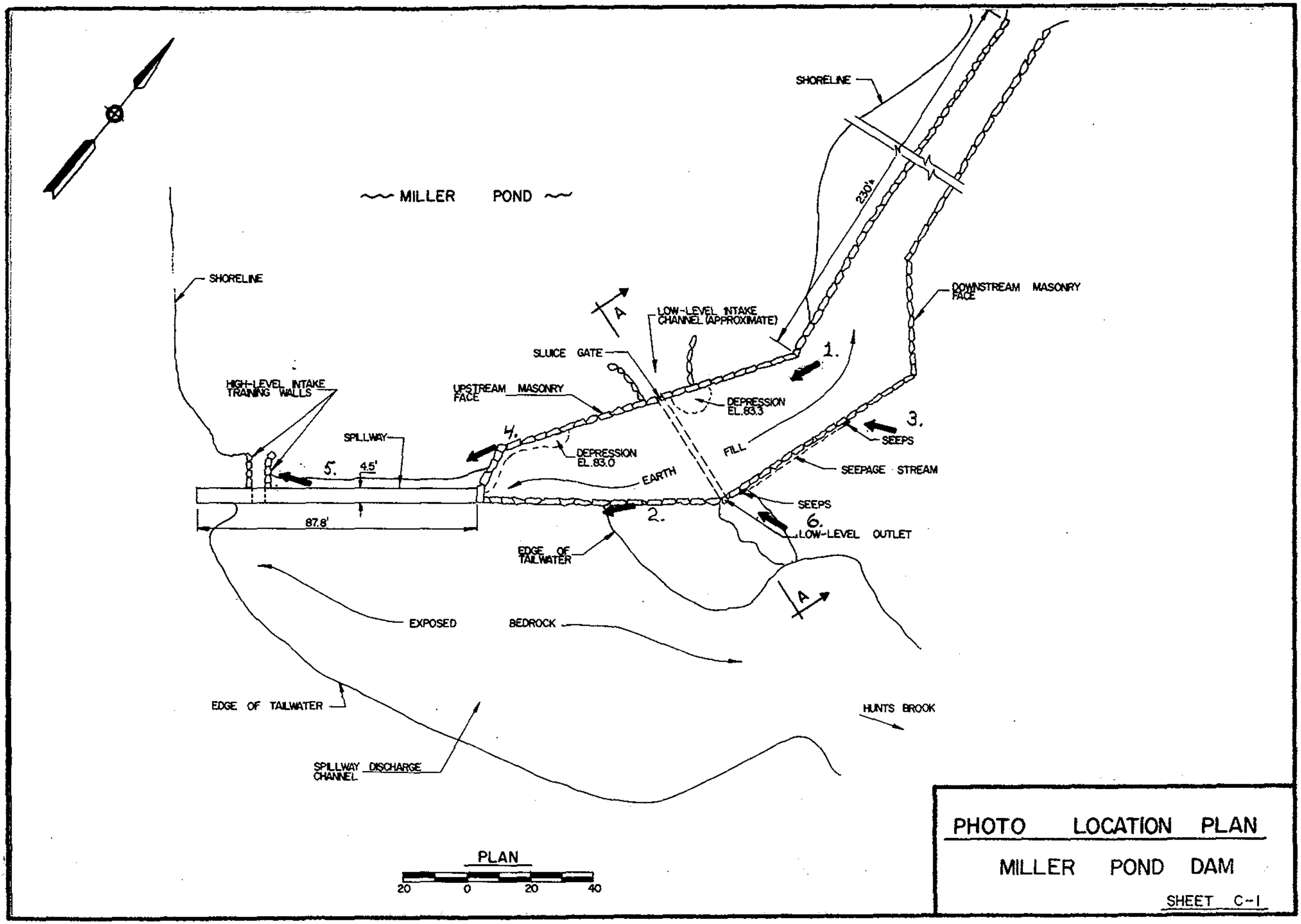


PHOTO LOCATION PLAN

MILLER POND DAM

SHEET C-1

Photo 1 - Crest of dam. Note erosion on crest and displacement of masonry on upstream face. (3/20/80).

Photo 2 - Spillway crest and discharge channel. Note high velocity flow along toe of dam (3/20/80).

Photo 3 - One of several seeps located approximately 50 feet to the left of the low-level outlet (3/20/80).

Photo 4 - Upstream slope of spillway (3/20/80).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Miller Pond Dam Hunts Brook Waterford, Conn.
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 785 KA DATE May '80 PAGE C-2

Photo 5 - Deteriorated right training wall of high-level outlet at right end of spillway (3/20/80).

Photo 6 - Downstream end of low-level outlet culvert (3/20/80).

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Miller Pond Dam Hunts Brook Waterford, Conn.
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 785 KA DATE May '80 PAGE C-3

APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS

PROBABLE MAXIMUM FLOOD (PMF) DETERMINATION

DRAINAGE AREA—
THE TOTAL DRAINAGE AREA FOR MILLER POND = 10.5 sq. mi
THIS WAS OBTAINED BY ACTUAL MEASUREMENT FROM USGS QUADRANGLE MAPS.

WATERSHED CLASSIFICATION — "ROLLING"
THIS CLASSIFICATION IS ASSIGNED BY EXAMINING THE USGS QUADRANGLE MAPS AND A VISUAL OBSERVATION OF SOME OF THE TERRAIN. EVEN THOUGH, SOME PARTS OF THIS WATERSHED IS "MOUNTAINOUS" AND SOME PARTS ARE FAIRLY FLAT. THE MAJORITY OF THE WATERSHED IS "ROLLING".

PMF PEAK INFLOW—
FROM CORPS OF ENGINEERS DECEMBER 1977 MAXIMUM PROBABLE FLOOD PEAK FLOW RATES GUIDE CURVE FOR A DRAINAGE AREA OF 10.5 SQ. MILES,
PEAK FLOW RATE = 1640 CFS/SQ. MILE
∴ PMF PEAK INFLOW = 10.5 X 1640
= 17,220 CFS.

SIZE CLASSIFICATION—
FOR THE PURPOSE OF DETERMINING PROJECT SIZE, THE MAXIMUM STORAGE ELEVATION IS CONSIDERED EQUAL TO THE TOP OF DAM ELEVATION.

TOP OF DAM ELEVATION	= 83.5* NGVD
ELEVATION OF THE TOE OF DAM AT LOWEST POINT	64.0 NGVD
∴ HEIGHT OF DAM	19.5 FEET

*THE WATER SURFACE ELEVATION OF 80 MSL FOR THE POND SHOWN ON THE USGS MONTVILLE QUADRANGLE MAP (REV. 1970) IS ASSUMED TO BE THE SPILLWAY CREST ELEVATION ON NATIONAL GEODETIC VERTICAL DATUM (NGVD) AND ALL OTHER ELEVATIONS ARE REFERENCED TO THIS ASSUMED ELEVATION

PLANIMETERING FROM USGS MAP FOR POND SURFACE AREAS
AT EL. 80 = 76.7 ACRES
AT EL. 85 = 95 ACRES (APPROXIMATE)
AT EL. 83.5 (TOP OF DAM) = 89.5 ACRES

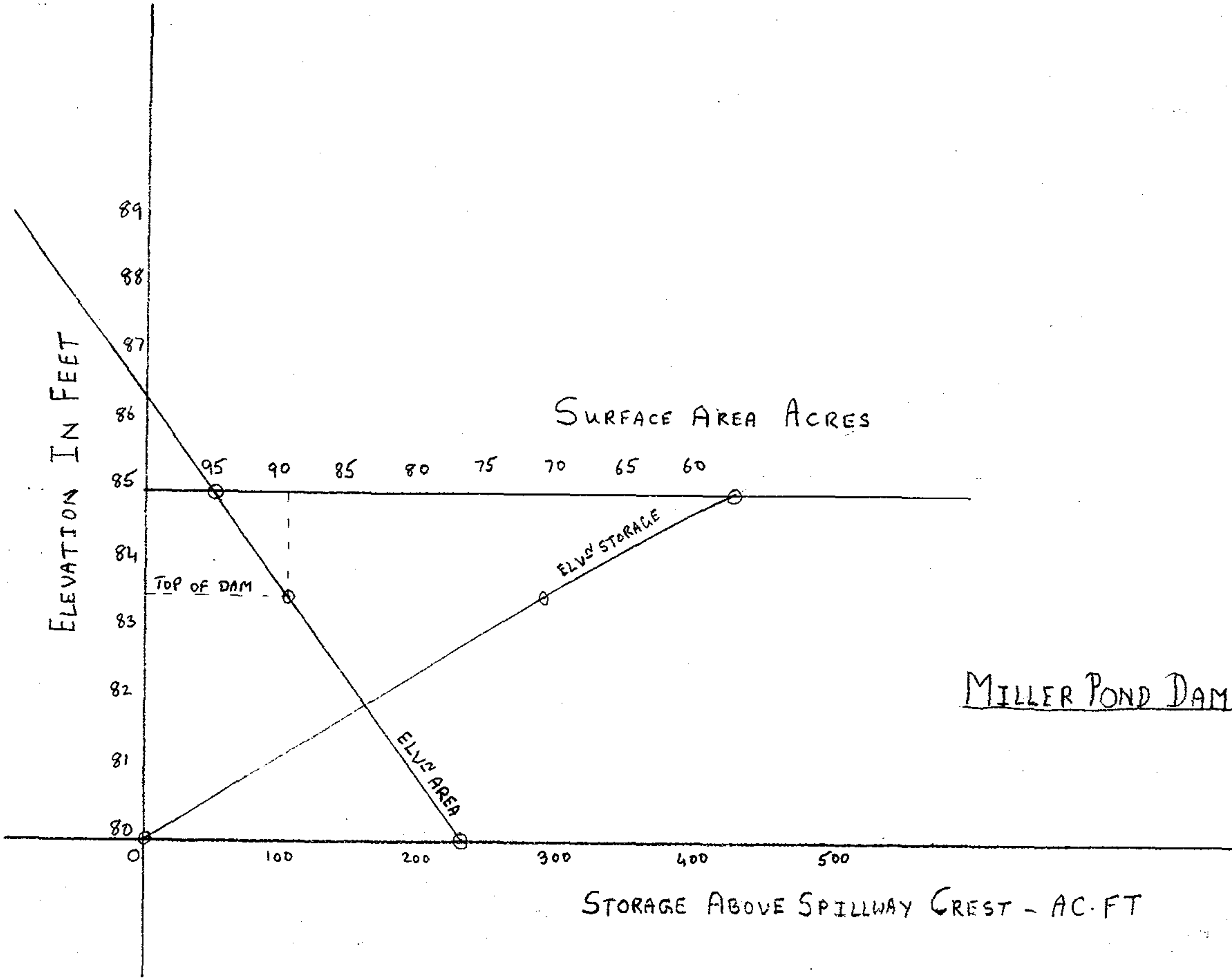
A STAGE-POND AREA CURVE IS PLOTTED (SHEET 3)
AVERAGE POND AREA BETWEEN SPILLWAY CREST = 83.1 ACRES
AND TOP OF DAM.
∴ STORAGE BETWEEN SPILLWAY CREST AND TOP OF DAM
$= 3.5 \times 83.1 = \underline{291}$ AC.FT.

ESTIMATED STORAGE BELOW SPILLWAY CREST
$= \frac{1}{3} \times 76.7 \times 16 = \underline{409}$ AC.FT.
(EL. 80 − EL. 64 = 16')

∴ <u>MAXIMUM IMPOUNDMENT TO TOP OF DAM</u> = 291 + 409 = <u>700 AC.FT.</u>

A STAGE-STORAGE CURVE IS PLOTTED (SHEET 3)
THUS, ACCORDING TO CORPS OF ENGINEERS GUIDELINES
TABLE 1, THE MILLER POND DAM IS CLASSIFIED <u>SMALL</u>
BASED UPON THE STORAGE CAPACITY OF 700 AC.FT.
($<$ 1000 AND $\geq$ 50) AND THE HEIGHT OF DAM
IS ONLY 19.5 FT.

HAZARD POTENTIAL — A CLASSIFICATION OF <u>HIGH</u>
HAZARD IS ASSIGNED BASED ON DAM BREACH ANALYSIS
AND RELATIVE LOCATIONS OF HOUSES AND OTHER
STRUCTURES DOWNSTREAM OF THE DAM. A DETAILED
DISCUSSION OF HAZARD POTENTIAL IS INCLUDED
IN THE BREACH ANALYSIS SECTION OF APPENDIX-D.



SHEET 3 OF 19
MA 7/9/80
7/10/80

SELECTION OF TEST FLOOD—

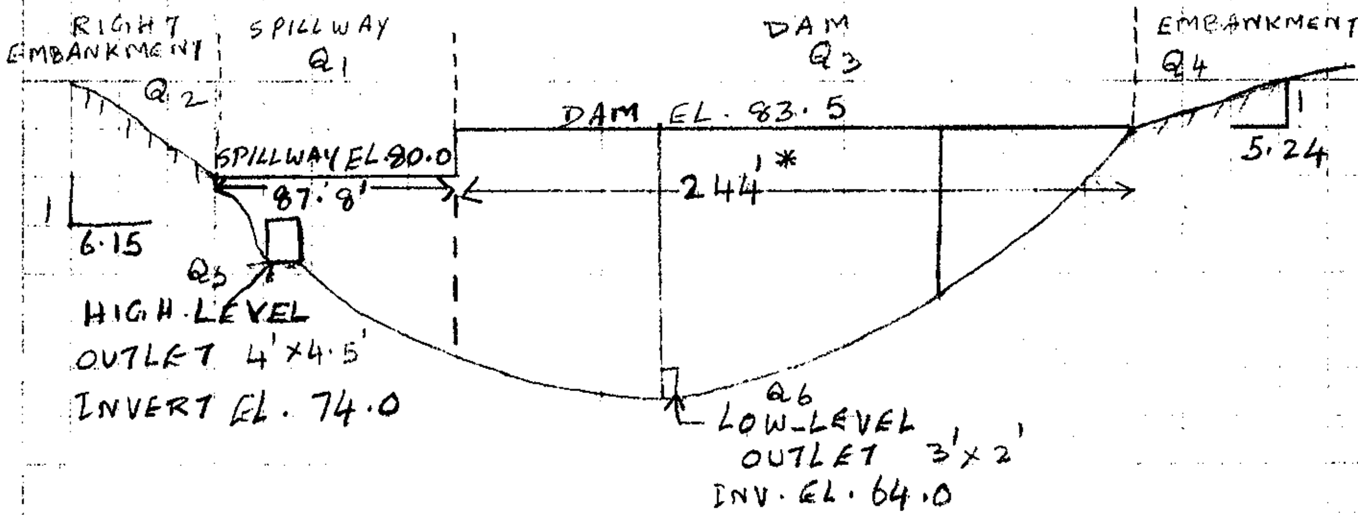
FOR THE SMALL SIZE AND HIGH HAZARD POTENTIAL CLASSIFICATION, TABLE 3 OF CORPS OF ENGINEERS RECOMMENDED GUIDELINES, THE TEST FLOOD COULD BE IN THE $\frac{1}{2}$ PMF TO PMF RANGE. BASED ON THE INVOLVED DOWNSTREAM RISK POTENTIAL WHICH IS CONSIDERED TO BE AT THE LOWER END OF THE HIGH HAZARD CLASSIFICATION SCALE A TEST FLOOD = $\frac{1}{2}$ PMF IS SELECTED.

TEST FLOOD PEAK INFLOW FOR $\frac{1}{2}$ PMF $= \frac{1}{2} \times 17{,}220$ CFS
$= 8{,}610$ CFS

NOTE: SURCHARGE STORAGE ROUTING IS ALSO PERFORMED FOR FULL PMF.

PROJECT NON FEDERAL DAM INSPECTION PROJECT NO. 80-10-10 SHEET 5 OF 19
NEW ENGLAND DIVISION COMPUTED BY MA DATE 4/28/80
MILLER POND DAM CHECKED BY EK DATE 4/29/80

COMPOSITE DISCHARGE RATING CURVE



POTENTIAL OVERFLOW PROFILE

(*HORIZONTAL PROJECTION OF THE LENGTH OF THE DAM NORMAL TO FLOW ON A VERTICAL PLANE)

THE OUTFLOW CAPACITIES OF VARIOUS SECTIONS ARE CALCULATED AND TABULATED ON SHEET 7.

THE SPILLWAY IS OF STONE MASONRY CONSTRUCTION AND OVERFLOW $Q_1 = CL\,H^{3/2}$, WHERE $C = 2.8$ AND $L = 87.8$ FT. A LOWER VALUE OF C IS CHOSEN BECAUSE OF IRREGULAR CREST CONDITION

THE OVERFLOW CAPACITY OF THE RIGHT EMBANKMENT IS CALCULATED BY, $Q_2 = \frac{2}{3} \times C \times L \times H^{3/2}$
FOR $C = 2.7$ AND AVERAGE SLOPE OF 1^V TO 6.15^H, $Q_2 = 11.07 \times H^{5/2}$

THE OVERFLOW CAPACITY OF THE DAM IS CALCULATED BY, $Q_3 = C \times L \times H^{3/2}$, WHERE BECAUSE OF IRREGULAR SHAPE C IS ASSUMED TO BE 2.6 AND THE EFFECTIVE LENGTH OF THE DAM IS 244 FT, AND

THEREFORE $Q_3 = 2.6 \times 244 \times H^{3/2}$

$= 634.4\ H^{3/2}$

THE OVERFLOW CAPACITY OF THE LEFT EMBANKMENT IS CALCULATED BY
$Q_4 = \frac{2}{3} \times C \times L \times H^{3/2}$,
FOR $C = 2.7$ AND AVERAGE SLOPE OF 1^V to 5.24^H,
$Q_4 = 9.43 H^{5/2}$.

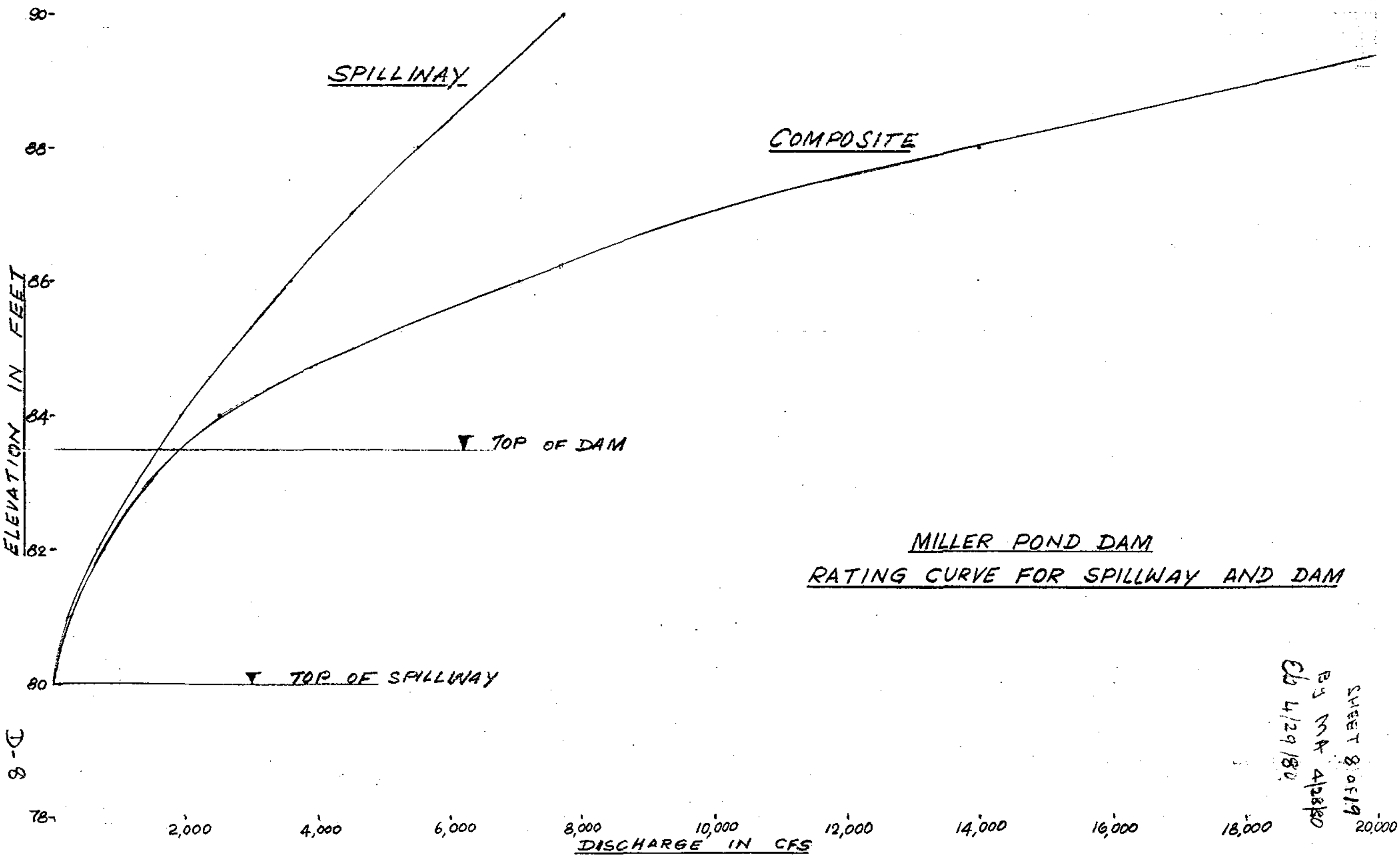
THE OUTFLOW CAPACITY OF THE HIGH-LEVEL OUTLET IS OBTAINED BY $Q_5 = 0.6 \times A \times \sqrt{2gH}$, WHERE $A = 4' \times 4.5'$, INVERT EL. 74.0 AND ELV$^{\underline{N}}$ OF CENTER OF THE OUTLET 76.0, $Q_5 = 86.4 \times H^{1/2}$

THE OUTFLOW CAPACITY OF THE LOW-LEVEL OUTLET IS OBTAINED BY $Q_6 = C A \sqrt{2gH}$, WHERE $A = 3' \times 2'$, AND ELEVATION OF THE CENTER OF THE OUTLET IS 65.5 AND $C = 0.6$ ASSUMED

TABULATION FOR DISCHARGE RATING CURVE

	ELVN	SPILLWAY $Q_1=245.8H^{3/2}$ L = 87.8' C.ELVN = 80.0	Q_2 $11.07 \times H^{5/2}$ RT. EMBANKMENT	DAM $Q_3=634.4 \times H^{3/2}$ L = 244' C.EL = 83.5	Q_4 $9.43 \times H^{5/2}$ LEFT EMBANKMENT.	TOTAL Q CFS
	78.0	0	0	0	0	0
SP. CR.	80.0	0	0	0	0	0
	81.0	246	11	0	0	257
	82.0	695	63	0	0	758
	83.0	1276	173	0	0	1449
DAM CREST	83.5	1610	254	0	0	1864
	84.0	1967	354	222	2	2545
	85.0	2749	619	1165	26	4559
	86.0	3613	976	2508	93	7190
POOL @ TEST FLOOD	86.2	3795	1060	2814	113	7782
	88.0	5563	2004	6056	405	14,028
PMF	88.48	6070	2320	7055	525	15,970
	90.0	7774	3479	10,513	1016	22,782

NOTE: THE TOTAL CAPACITY Q DOES NOT INCLUDE THE DISCHARGE CAPACITY OF THE HIGH-LEVEL OUTLET, BECAUSE OF SMALL QUANTITIES, (FOR POOL AT TOP OF DAM $Q_5 \cong 235$ CFS.)
SIMILARLY, THE DISCHARGE CAPACITY OF THE LOW-LEVEL OUTLET Q_6 IS NEGLECTED BECAUSE OF SMALL QUANTITIES (FOR POOL AT TOP OF DAM $Q_6 \cong 130$ CFS)
WITH THE ABOVE DATA, DISCHARGE RATING CURVES ARE PLOTTED ON SHEET 8.


MILLER POND DAM
RATING CURVE FOR SPILLWAY AND DAM
SPILLWAY
COMPOSITE
TOP OF DAM
TOP OF SPILLWAY
ELEVATION IN FEET
DISCHARGE IN CFS
90
88
86
84
82
80
78
2,000
4,000
6,000
8,000
10,000
12,000
14,000
16,000
18,000
20,000
SHEET 8 OF 19
By MA 4/28/80
Ck 4/29/80

PROJECT NONFEDERAL DAM INSPECTION PROJECT NO. 80-10-10 SHEET 9 OF 19
NEW ENGLAND DIVISION COMPUTED BY MA DATE 4/28/80
MILLER POND DAM CHECKED BY EB DATE 4/29/80

DETERMINATION OF PEAK OUTFLOW
FOR $\frac{1}{2}$ PMF TEST FLOOD PEAK INFLOW OF 8610 CFS:

TRIAL #1:
THE PMF HAS 19" OF RUN-OFF FROM DRAINAGE AREA
∴ $\frac{1}{2}$ PMF HAS 9.5" OF RUN-OFF FROM THE DRAINAGE AREA.

FOR A DRAINAGE AREA OF 10.5 SQ. MILES.
AND A HEAD OF 3.5 FT

AVAILABLE SURCHARGE STORAGE UPTO TOP OF DAM
(AVERAGE POND AREA 83.1 ACRES)

$$= \frac{83.1 \times 3.5 \times 12}{10.5 \times 640}$$

= 0.52 INCHES OF RUN OFF FROM DRAINAGE AREA.

$$\frac{\text{POND SURCHARGE STORAGE}}{\text{INFLOW RUN OFF VOLUME}} = \frac{0.52}{9.5} = 0.055$$

REFERRING TO FIGURE 17-11 "TYPICAL SHORTCUT METHOD OF RESERVOIR FLOOD ROUTING" IN SCS NEH SECTION 4 AUGUST 1972, FOR $\frac{\text{POND SURCHARGE STORAGE}}{\text{INFLOW RUNOFF VOLUME}}$ OF 0.055 THE GUIDE CURVES GIVES

$$\frac{\text{OUTFLOW PEAK RATE}}{\text{INFLOW PEAK RATE}} = 0.98$$

∴ OUTFLOW PEAK RATE $= 0.98 \times 8610$

$= 8438$ CFS.

TRIAL # 2 —

FROM THE COMPOSITE RATING CURVE, THE OUTFLOW OF 8438 CFS CORRESPONDS TO EL. 86.4
∴ SURCHARGE HEIGHT ABOVE THE SPILLWAY CREST (EL. 80.0) = 6.4 FT.
POND AREA IS 100 ACRES AT EL. 86.4
AVERAGE POND AREA 88.35 ACRES
VOLUME OF SURCHARGE STORAGE ($STOR_1$) $= \frac{88.35 \times 6.4}{10.5 \times 640} \times 12 = 1.01''$ OF DRAINAGE AREA

PEAK OUTFLOW $Q_{P_2} = Q_{P_1}\left(1 - \frac{STOR_1}{9.5}\right)$

$= 8610\left(1 - \frac{1.01}{9.5}\right)$

$= 7695$ CFS.

TRIAL # 3 —

FROM THE COMPOSITE RATING CURVE, THE OUTFLOW OF 7695 CFS CORRESPONDS TO EL. 86.2
SURCHARGE HEIGHT ABOVE SPILLWAY CREST (EL. 80.0) = 6.2 FT.
POND AREA IS 99.4 ACRES @ EL. 86.2
VOLUME OF SURCHARGE STORAGE $= \frac{88.05 \times 6.2}{10.5 \times 640} \times 12 = 0.97$ INCHES OF DRAINAGE AREA.
(AVERAGE LAKE AREA 88.05 ACRES)

∴ PEAK OUTFLOW $Q_{P_2} = 8610\left(1 - \frac{0.97}{9.5}\right)$

$= \underline{7730 \text{ CFS}}$*

THIS OUTFLOW CORRESPONDS TO A MAXIMUM POOL ELEVATION = 86.2
∴ MAXIMUM SURCHARGE HEIGHT ABOVE SPILLWAY CREST (EL. 80.0) = $\underline{6.2}$ FT.

* THIS WAS CHECKED USING THE CORPS OF ENGINEERS GUIDELINES "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD.

PROJECT NON FEDERAL DAM INSPECTION PROJECT NO. 80-10-10 SHEET 11 OF 19
NEW ENGLAND DIVISION COMPUTED BY MA DATE 4/28/80
MILLER POND DAM CHECKED BY Eb DATE 4/29/80

NON-OVERFLOW SECTION OF THE DAM WOULD BE OVERTOPPED BY 2.7 FT.

(THE DIFFERENCE IN EL. 86.2 AND EL. 83.5)

THE CAPACITY OF THE SPILLWAY AT MAXIMUM POOL AS A PERCENT OF ROUTED TEST FLOOD OUTFLOW

$$= \frac{3795}{7730} = 49\%$$

AND CAPACITY OF THE SPILLWAY TO TOP OF DAM AS A PERCENT OF TEST FLOOD OUTFLOW

$$= \frac{1610}{7730} = 21\%$$

ROUTING IS ALSO DONE FOR PMF

DETERMINATION OF PEAK OUTFLOW
FOR PMF PEAK INFLOW OF 17,220 CFS

TRIAL #1
THE PMF HAS 19" OF RUN OFF FROM DRAINAGE AREA
AVAILABLE SURCHARGE STORAGE UPTO TOP OF DAM

= 0.52 INCHES OF RUN OFF FROM DRAINAGE AREA

$$\frac{\text{POND SURCHARGE STORAGE}}{\text{INFLOW RUN OFF VOLUME}} = \frac{0.52}{19} = 0.03$$

REFERING TO FIGURE 17-11 "TYPICAL SHORTCUT METHOD OF RESERVOIR FLOOD ROUTING" IN SCS NEH SECTION 4 AUGUST 1972

FOR 0.03 THE GUIDE CURVES GIVES

$$\frac{\text{OUTFLOW PEAK RATE}}{\text{INFLOW PEAK RATE}} = 0.99$$

$\therefore$ OUTFLOW PEAK RATE = 0.99 × 17,220
$\cong$ 17,000 CFS

TRIAL #2
FROM THE COMPOSITE RATING CURVE, THE OUTFLOW OF 17,000 CFS CORRESPONDS TO EL. 88.7
$\therefore$ SURCHARGE HEIGHT ABOVE THE SPILLWAY CREST (EL. 80.0) = 8.7 FT.

POND AREA AT EL. 88.7 = 108.5 ACRES
AVERAGE POND AREA = 92.6 ACRES

VOLUME OF SURCHARGE STORAGE ($STOR_1$)

$$= \frac{92.6 \times 8.7}{10.5 \times 640} \times 12 = 1.44'' \text{ OF D.A}$$

PEAK OUTFLOW $QP_2 = QP_1\left(1 - \frac{STOR_1}{19}\right) = 17{,}220\left(1 - \frac{1.44}{19}\right)$
$\cong 15{,}900$ CFS

TRIAL # 3 –

FROM THE COMPOSITE RATING CURVE, THE OUTFLOW OF 15,900 CFS CORRESPONDS TO EL. 88.45.
SURCHARGE HEIGHT ABOVE SPILLWAY CREST (EL.80.0) = 8.45 FT.
POND AREA AT EL. 88.45 = 107.75 ACRES.
VOLUME OF SURCHARGE STORAGE ($STOR_1$) $= \frac{92 \times 8.4}{10.5 \times 640} \times 12 = 1.38''$ OF D.A.
(AVERAGE POND AREA $\cong$ 92 ACRES)

∴ PEAK OUTFLOW $QP_2 = 17{,}220\left(1 - \frac{1.38}{19}\right) \cong$ 15,970 CFS

THIS OUTFLOW CORRESPONDS TO A MAXIMUM POOL ELEVN = 88.48
∴ MAXIMUM SURCHARGE HEIGHT ABOVE SPILLWAY CREST (EL. 80)
= 8.48 FT.

NON-OVERFLOW SECTION OF THE DAM WOULD BE OVERTOPPED BY (THE DIFFERENCE IN EL. 88.48 AND EL. 83.5) = 4.98 FT

THE CAPACITY OF THE SPILLWAY TO MAX^{M} POOL AS A % OF PMF PEAK FLOOD OUTFLOW $= \frac{6070}{15{,}970} = 38\%$

AND CAPACITY OF THE SPILLWAY TO TOP OF DAM AS A PERCENT OF PMF PEAK FLOOD OUTFLOW

$$= \frac{1610}{15{,}970} = 10\%$$

BREACH ANALYSIS

DOWNSTREAM FAILURE HAZARD —

PEAK FLOOD AND STAGE IMMEDIATELY DOWNSTREAM FROM DAM PER CORPS OF ENGINEERS DAM FAILURE GUIDELINES —

BREACH OUTFLOW $Q_b = \frac{8}{27} W_b \sqrt{g} \cdot Y_0^{3/2}$

BREACH WIDTH W_b:

THE EFFECTIVE LENGTH OF THE DAM AT MID-HEIGHT = 173 FT.

40% OF 173 = 69.2 FT. USE W_b = 69 FT.

Y_0: USING POOL ELEVATION AT TOP OF DAM (EL. 83.5) TO COMPUTE PEAK FAILURE OUTFLOW. HEIGHT AT TIME OF FAILURE IS DIFFERENCE BETWEEN EL. 83.5 AND EL. 64.0

Y_0 = 19.5 FT

BREACH OUTFLOW $Q_b = \frac{8}{27} \times 69 \sqrt{32.2} \times (19.5)^{3/2}$

$\cong$ 10,000 CFS.

FOR POOL AT TOP OF DAM —

SPILLWAY FLOW PRIOR TO FAILURE = 1610 CFS.

FLOW OVER THE RIGHT EMBANKMENT PRIOR TO FAILURE = 254 CFS

∴ TOTAL PEAK FAILURE OUTFLOW Q_{P_1} = 10,000 + 1610 + 254 = 11,864 CFS

SAY, 12,000 CFS.

ESTIMATED FAILURE FLOOD DEPTH $\cong \frac{2}{3} Y_0$

IMMEDIATELY D/S FROM DAM $\cong \frac{2}{3} \times 19.5 \cong$ 13 FT.

FLOOD STAGE AND DEPTH D/S REACHES

AN EXAMINATION OF THE D/S CONDITIONS INDICATES THAT THE MAJORITY OF THE POTENTIAL FLOOD FLOW REACH IS QUITE NARROW AND STEEP; THEREFORE A VERY SMALL QUANTITY OF FLOOD VOLUME WOULD BE STORED. HENCE, THE FOLLOWING ANALYSIS IS BASED ON THE ASSUMPTION THAT NO ATTENUATION OF THE FLOOD VOLUME TAKES PLACE IN THE D/S REACHES OF THE DAM.

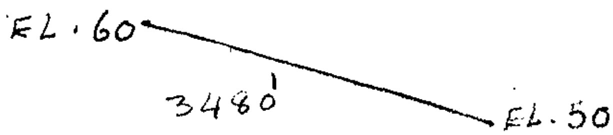
INITIAL IMPACT AREA (BLOOMINGDALE Rd. VICINITY) —

BY USING MANNING'S EQUATION —

$$Q = A \times \frac{1.486}{n} (R)^{2/3} \times (S)^{1/2}$$

THE BED SLOPE S IS DETERMINED FROM USGS QUADRANGLE MAP — A DROP IN ELEVATION OF 10 FT. IN 3480 FT.

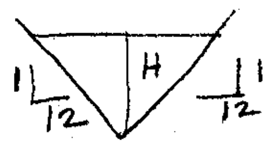
$\therefore S = 0.0029$



FROM AN OBSERVATION OF THE SITE AS WELL AS USGS MAP, THE SLOPES OF BOTH SIDES OF THE CHANNEL ARE ASSUMED TO BE 1^V TO 12 H

$A = 12H^2$, $P = H\sqrt{145} + H\sqrt{145} = 24.08H$

$R = \frac{A}{P} = \frac{12H^2}{24.08H} = 0.498H$



$\therefore$ IN MANNING'S EQUATION —

FOR TOTAL PEAK FAILURE OUTFLOW OF 12,000 CFS AND $n = 0.025$ —

$$12{,}000 = 12H^2 \times \frac{1.486}{0.025} \times (0.498H)^{2/3} \times (0.0029)^{1/2}$$

$$= 24.14H^{8/3}$$

FLOOD DEPTH H JUST PRIOR TO REACHING BLOOMINGDALE ROAD $\cong$ 10.3 FT.

AND FLOOD STAGE $\cong$ 61.5 (APPROXIMATE CHANNEL BED EL 51.2)

SIMILARLY, USING THE ABOVE PROCEDURE, FOR

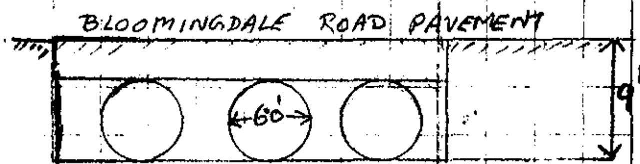
PROJECT NON FEDERAL DAM INSPECTION PROJECT NO. 80-10-10 SHEET 16 OF 19
NEW ENGLAND DIVISION COMPUTED BY MA DATE 4/29/80
MILLER POND DAM CHECKED BY SL DATE 4/30/80

PREFAILURE FLOW 1864 CFS,
PREFAILURE FLOOD DEPTH = 5.1 FT
AND PREFAILURE STAGE = 56.3

THUS, THE RAISE IN STAGE AFTER DAM FAILURE AT INITIAL IMPACT AREA

$$\Delta Y_1 = 61.5 - 56.3 = 5.2\ FT$$

AT THIS IMPACT AREA THERE IS A CULVERT CONSISTING OF THREE 72" CONCRETE PIPES AND ITS CAPACITY IS EXAMINED WITH FULL FLOW AND FREE OUTFALL CONDITION,



FOR INLET CONTROL $\frac{\text{HEADWATER } HW}{\text{DIAMETER } d} = \frac{9.00}{6.0}$

$= 1.5$

AND USING U.S. BUREAU OF PUBLIC ROADS JANUARY '63 NOMOGRAPH FOR HEADWATER SCALE NO. 2, REVISED MAY 1964,
DISCHARGE THROUGH EACH PIPE = 350 CFS.
THUS, UNDER THE ABOVE CONDITIONS TOTAL DISCHARGE THROUGH ALL THE THREE PIPES = 3X350 = 1050 CFS, WHICH IS ONLY 9% OF THE TOTAL PEAK FAILURE OUTFLOW OF 12,000 CFS.

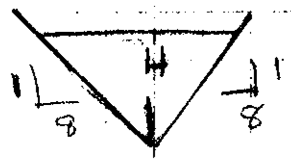
THE HOUSE NORTH OF THE BROOK ADJACENT TO BLOOMINGDALE ROAD IS APPROXIMATELY 8 FT. ABOVE THE CHANNEL BED. SINCE THE FLOOD DEPTH AT DAM FAILURE IS ESTIMATED TO BE 10.3 FT, THE 1 ST FLOOR OF THE HOUSE WILL BE FLOODED BY 2.3± FT OF WATER. THE SITUATION WOULD BE FURTHER AGGRAVATED BECAUSE OF THE INADEQUATE CAPACITY OF THE CULVERT.

TWO ADDITIONAL HOMES SOUTH OF THE BROOK WOULD BE SIMILARLY IMPACTED WITH $2^{\pm}$ FT OF FLOOD WATER.

SECONDARY IMPACT AREA (OLD NORWICH ROAD)

THE CHANNEL BED SLOPE, S IS DETERMINED BY FIELD INFORMATION AND USGS MAP – S = 0.0093

FROM AN OBSERVATION OF THE SITE AND AN EXAMINATION OF USGS MAP, THE SLOPES OF BOTH SIDES OF THE CHANNEL ARE ASSUMED TO BE 1^V to 8^H



USING MANNING'S EQUATION

$$Q = A \times \frac{1.486}{n} (R)^{2/3} (S)^{\frac{1}{2}}$$

$$A = 8H^2, \quad P = H\sqrt{65} + H\sqrt{65} = 16.12H$$

$$R = \frac{A}{P} = \frac{8H^2}{16.12H} = 0.496H$$

FOR TOTAL PEAK FAILURE OUTFLOW OF 12,000 CFS AND n = .025

$$12{,}000 = 8H^2 \times \frac{1.486}{.025} \times (.496H)^{2/3} \times (.0093)^{\frac{1}{2}}$$

$$= 28.72H^{8/3}$$

∴ FLOOD DEPTH JUST PRIOR TO REACHING OLD NORWICH ROAD ≅ 9.6 FT.
AND FLOOD STAGE ≅ 12.6 (APPROXIMATE CHANNEL BED. EL. 3)

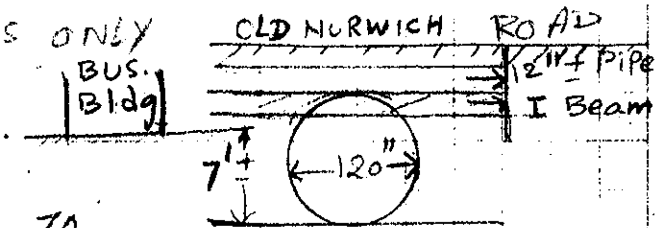
SIMILARLY, USING THE ABOVE PROCEDURE, FOR A PREFAILURE FLOW OF 1864 CFS,

PREFAILURE FLOOD DEPTH = 4.8 FT.
AND PREFAILURE STAGE = 7.8

AND THE RAISE IN STAGE AFTER DAM FAILURE AT SECONDARY IMPACT AREA $\Delta Y_2 = 12.6 - 7.8 = \underline{4.8}$ FT.

AT THIS IMPACT AREA THERE IS A 120" CULVERT AND ITS CAPACITY IS EXAMINED SIMILARLY AS THE CULVERT ON INITIAL IMPACT AREA WITH $\frac{Hw}{d} = \frac{120}{120} = 1$

AND Q = 800 CFS WHICH IS ONLY 7% OF THE TOTAL PEAK FAILURE OUTFLOW OF 12,000 CFS.



THE BUILDING ADJACENT TO THE CULVERT AND CONTAINING SEVERAL BUSINESSES, IS 3'± BELOW THE CROWN AND THEREFORE WILL BE FLOODED BY 2.6± FEET OF WATER. A PORTION OF OLD NORWICH Rd, WHICH IS A WELL TRAVELED ROADWAY WOULD BE SUBMERGED AND IT IS LIKELY THAT TWO DWELLINGS ADJACENT TO THIS CULVERT WOULD ALSO BE IMPACTED WITH 2± FEET OF FLOOD WATER.

THUS, THE PEAK FAILURE OUTFLOW WOULD RESULT IN FLOOD OF A MAGNITUDE THAT WOULD IMPACT AT LEAST FIVE HOUSES, A BUILDING CONTAINING SEVERAL BUSINESSES AND THREE CULVERTS. IT IS REPORTED THAT ONE OF THESE CULVERTS, 2200± FT. D/S OF THE DAM WAS WASHED OUT DUE TO A RECENT FLOODING.

BASED ON ABOVE ANALYSIS, A HAZARD POTENTIAL OF HIGH MAGNITUDE IS CONSIDERED LIKELY.

SUMMARY - HYDRAULIC/HYDROLOGIC COMPUTATIONS

TEST FLOOD PEAK INFLOW $\frac{1}{2}$PMF 8610 CFS
(Parallel computations have been performed for PMF Peak inflow and results are summarized below)

PERFORMANCE AT PEAK FLOOD CONDITIONS:	$\frac{1}{2}$PMF	PMF
PEAK INFLOW CFS	8610	17,220
PEAK OUTFLOW CFS	7730	15,970
SPILL CAP TO TOP OF DAM (EL. 83.5 NGVD) CFS	1610	1,610
SPILL CAP TO TOP OF DAM % OF PEAK OUTFLOW	21	10
SPILL CAP TO PEAK FLOOD ELVN. CFS	3,795	6,070
SPILL CAP. TO PEAK FLOOD ELVN. % OF PEAK OUTFLOW	49	38
PERFORMANCE:		
MAX. POOL ELEVATION NGVD	86.2	88.48
MAX. SURCHARGE HEIGHT ABOVE SPILL CREST FT.	6.2	8.5±
NON-OVERFLOW SECTION OF THE DAM OVERTOPPED FT	2.7	5±

DOWNSTREAM FAILURE CONDITIONS:	
PEAK FAILURE OUTFLOW CFS	12,000
FLOOD DEPTH IMMEDIATELY D/S FROM DAM	13 FT.
CONDITIONS AT THE INITIAL IMPACT AREA:	
ESTIMATED STAGE BEFORE FAILURE WITH 1864 CFS	56.3 NGVD
ESTIMATED STAGE AFTER FAILURE WITH 12,000 CFS	61.5 NGVD
ESTIMATED RAISE IN STAGE AFTER FAILURE Δy_1	5.2 FT.
CONDITIONS AT THE SECONDARY IMPACT AREA:	
ESTIMATED STAGE BEFORE FAILURE WITH 1864 CFS	7.8 NGVD
ESTIMATED STAGE AFTER FAILURE WITH 12,000 CFS	12.6 NGVD
ESTIMATED RAISE IN STAGE AFTER FAILURE Δy_2	4.8 FT.

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

March 1978

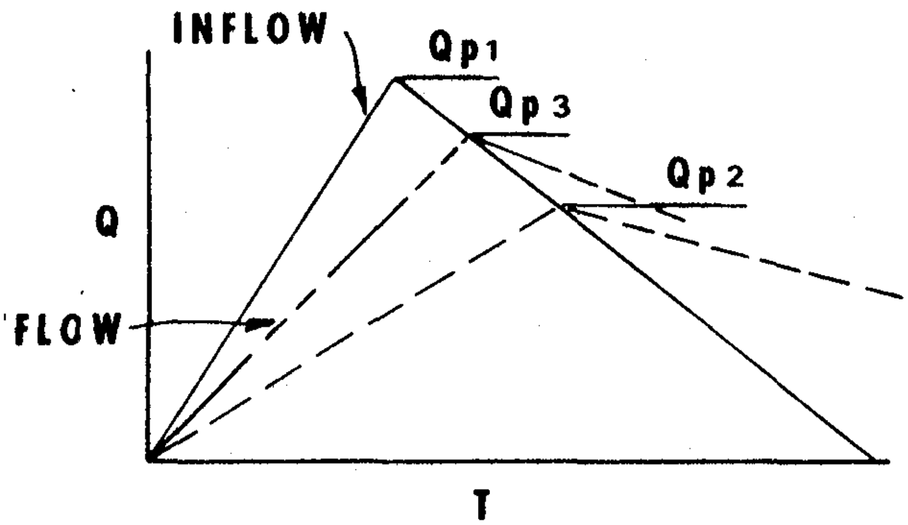
MAXIMJM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

MAXIMUM PROBABLE FLOWS
BASED ON TWICE THE
STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Qp_1) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Qp_1".

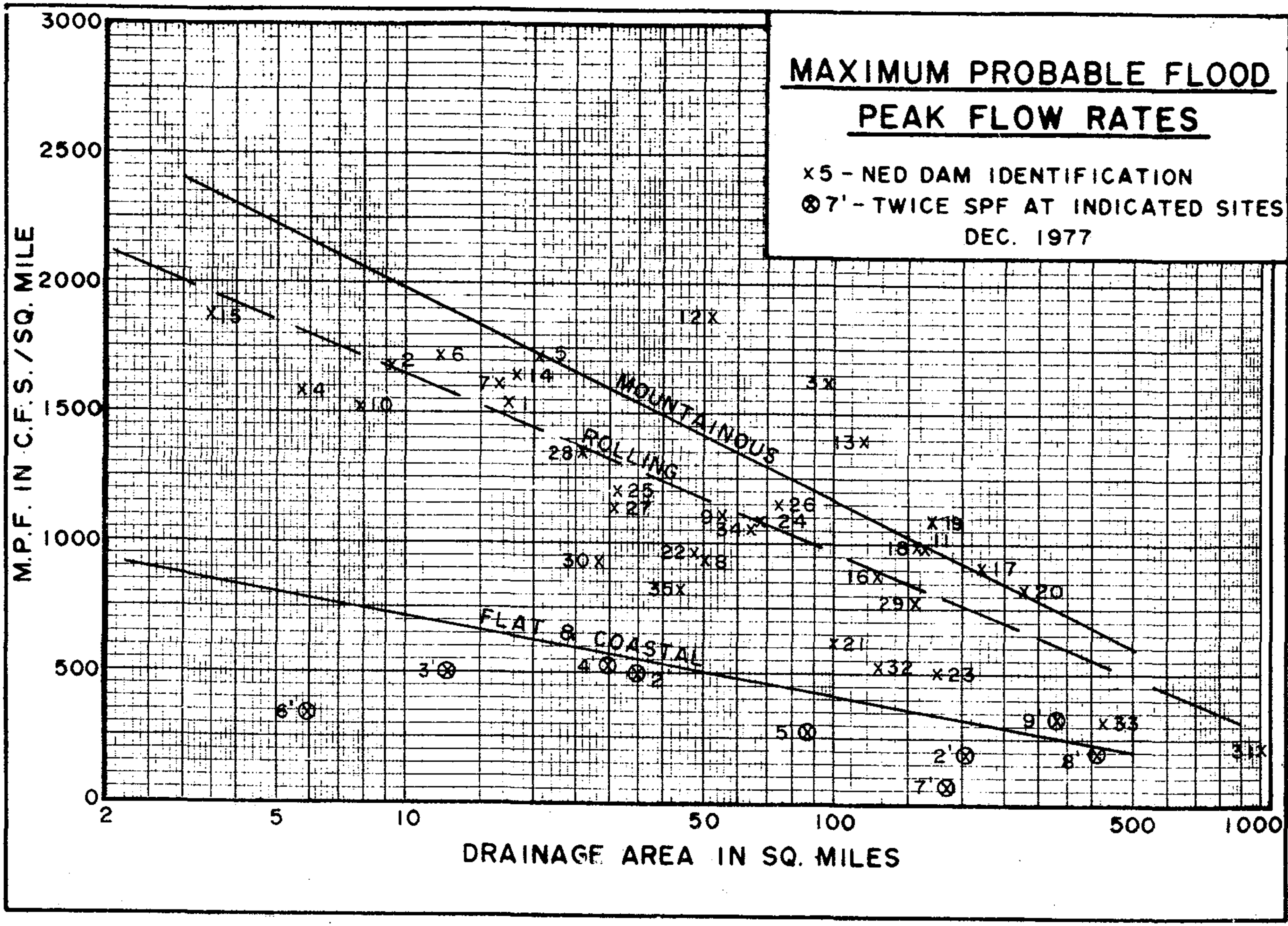
b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore:

$$Qp_2 = Qp_1 \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Qp_3".



MAXIMUM PROBABLE FLOOD
PEAK FLOW RATES
×5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SITES
DEC. 1977
MOUNTAINOUS
ROLLING
FLAT & COASTAL
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
3000
2500
2000
1500
1000
500
0
2
5
10
50
100
500
1000

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

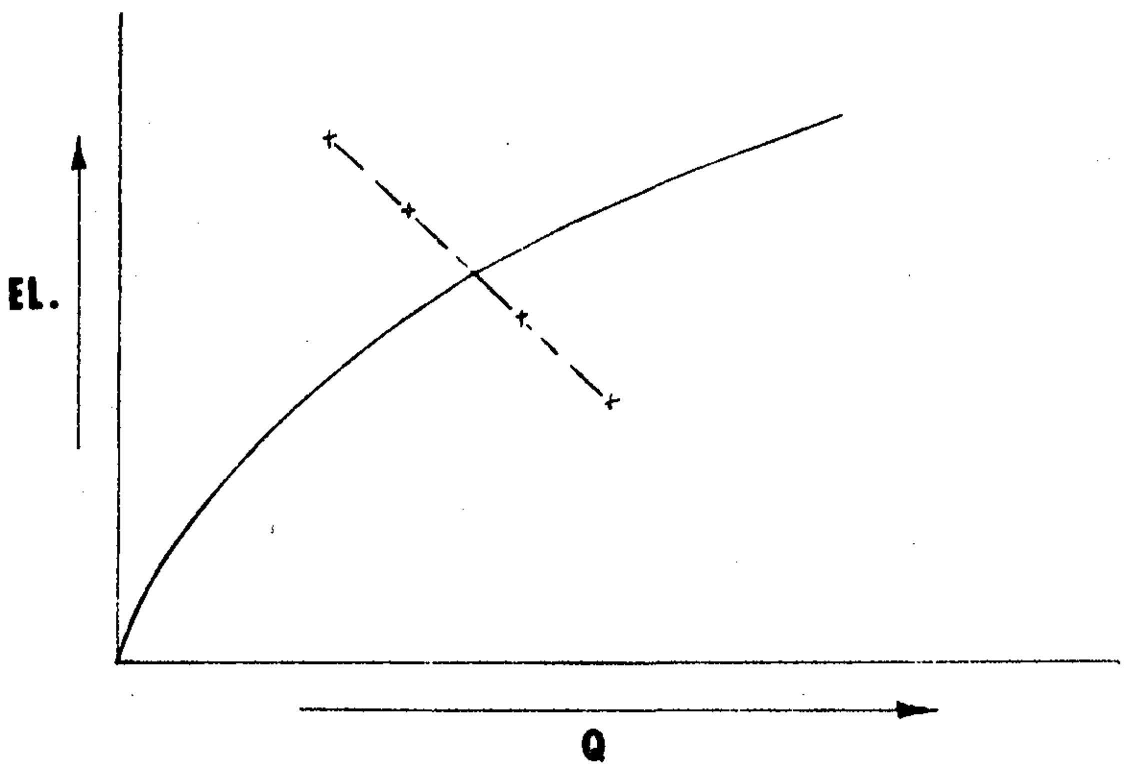
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

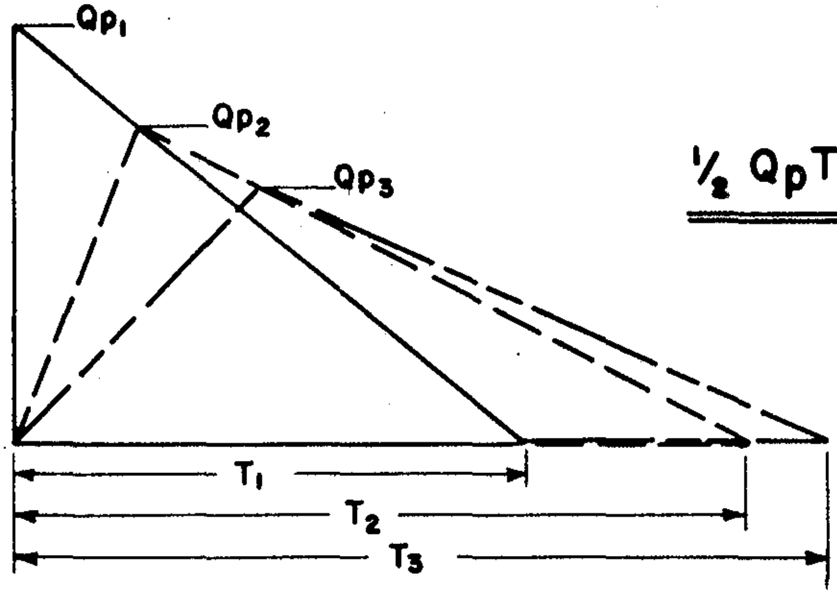
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19" R.O.

Q_{p2}	STOR	EL.



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\frac{1}{2} Q_p T = 12\,S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_o = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{AVG}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN THE NATIONAL INVENTORY OF DAMS

THAMES RIVER BASIN

WATERFORD, CONNECTICUT

MILLER POND DAM
00154

PHASE I INSPECTION REPORT

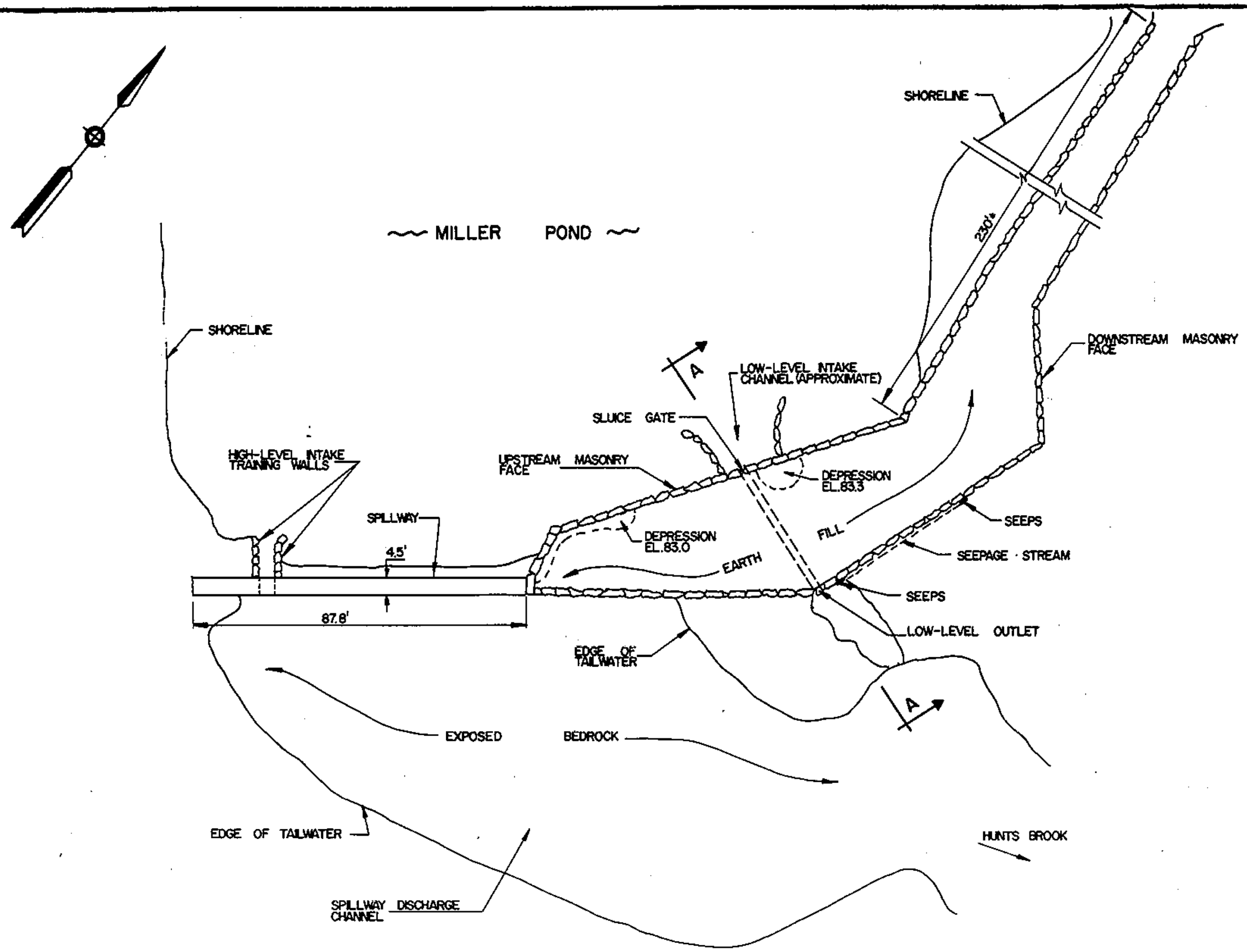
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

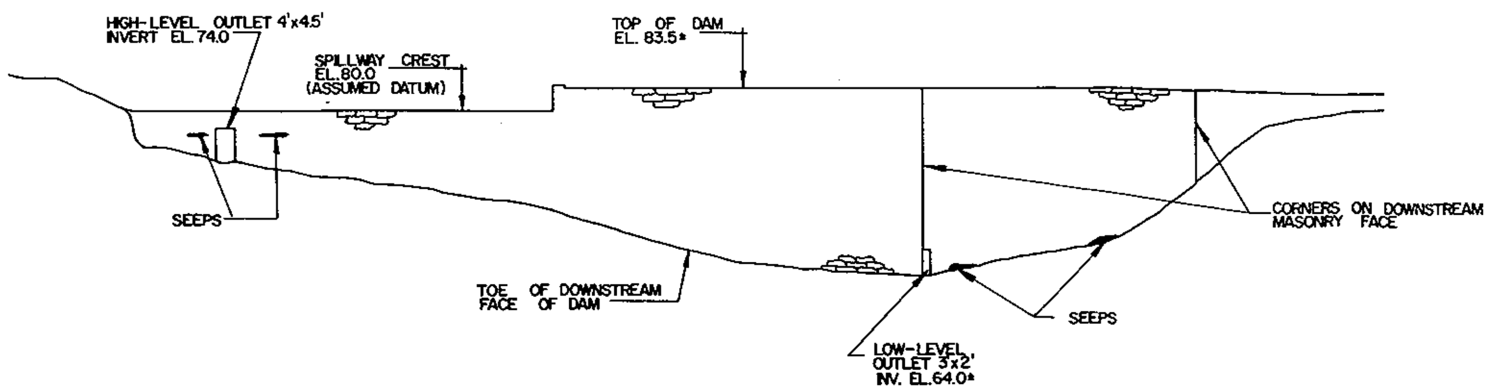
WALTHAM, MASS. 02154

AUGUST 1980



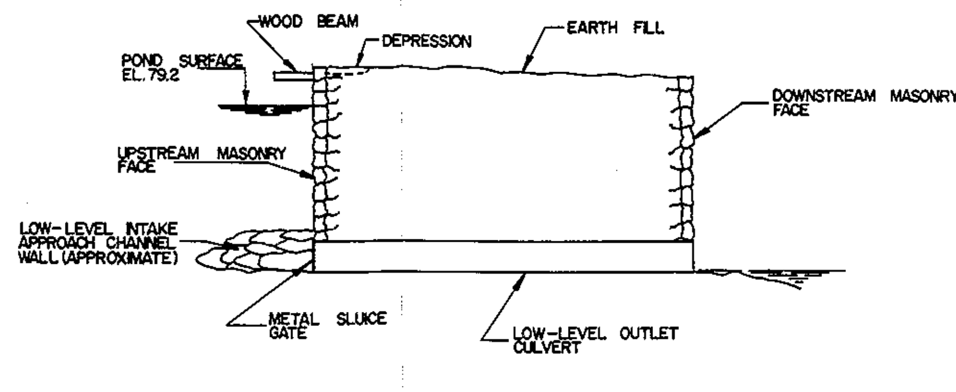
PLAN

20 0 20 40



ELEVATION

HORIZONTAL 20 0 20 40
VERTICAL 10 0 10 20



SECTION A-A

10 0 10 20

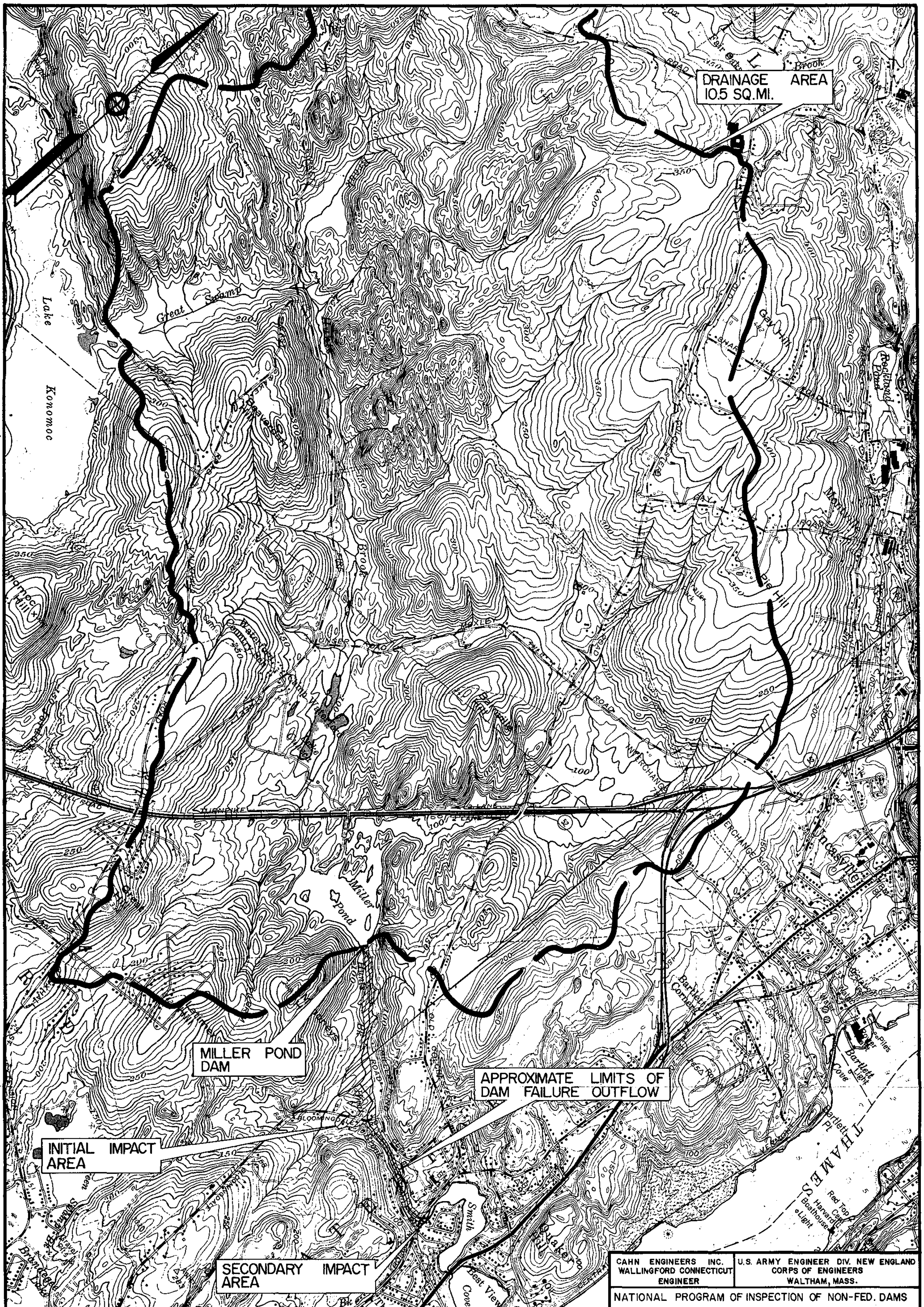
NOTES:

1. THIS PLAN WAS COMPILED FROM A CAHN ENGINEERS INSPECTION OF THE DAM DATED MARCH 26, 1980. DIMENSIONS SHOWN ARE APPROXIMATE. NOT ALL TOPOGRAPHIC AND/OR STRUCTURAL FEATURES ARE NECESSARILY IDENTIFIED.

2. NO ELEVATIONS WERE AVAILABLE FOR THE DAM, THEREFORE THE WATER SURFACE ELEVATION OF 80.0 FOR THE POND SHOWN ON THE U.S.G.S. MONTVILLE QUADRANGLE MAP WAS ASSUMED TO BE THE ELEVATION OF THE SPILLWAY CREST. ALL OTHER ELEVATIONS SHOWN ARE REFERENCED TO THE ASSUMED SPILLWAY CREST ELEVATION.

3. WATER SURFACE ELEVATIONS, SHORELINE AND TAILWATER CONFIGURATIONS ARE APPROXIMATE, AS OBTAINED DURING THE DAM INSPECTION ON MARCH 26, 1980.

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER		U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS			
PLAN, ELEVATION & SECTION			
MILLER POND DAM			
HUNTS BROOK		WATERFORD, CONNECTICUT	
DRAWN BY	CHECKED BY	APPROVED BY	SCALE: AS NOTED
K. Hartman	TJS	[illegible]	DATE: MAY 1980 — SHEET B-1


DRAINAGE AREA 10.5 SQ.MI.
MILLER POND DAM
APPROXIMATE LIMITS OF DAM FAILURE OUTFLOW
INITIAL IMPACT AREA
SECONDARY IMPACT AREA
U.S.G.S. QUADRANGLES
MONTVILLE 1970
UNCASVILLE 1970
CAHN ENGINEERS INC.
WALLINGFORD CONNECTICUT
ENGINEER
U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS
DRAINAGE AREA MAP
MILLER POND DAM
HUNTS BROOK
MONTVILLE, CONNECTICUT
DWN. BY
CKD. BY
TJS
APP. BY
SCALE: 1" = 2000'
DATE: MAY 1980
SHEET D-1
Lake Konomoc
Great Swamp
Miller Pond
Gay Hill
THAMES